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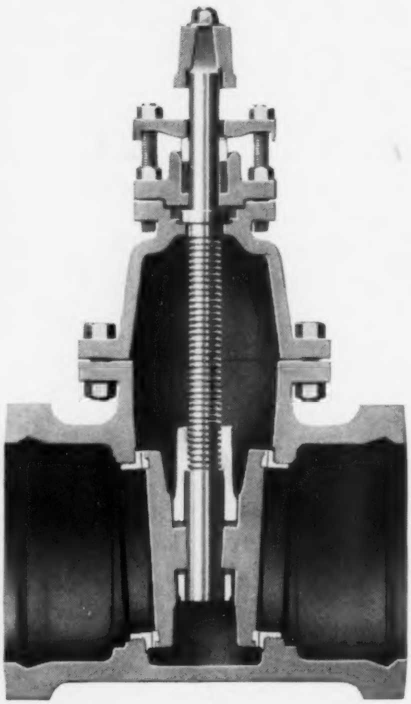
In this issue



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Contents

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Charles H. Capen, President 1952–1953

Journal
AMERICAN WATER WORKS ASSOCIATION
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Minimum Standards for Design, Construction, and Maintenance of a Public Water Distribution System

Michigan Section AWWA

Committee Statement

At the 1949 meeting of the Michigan Section of the American Water Works Assn. at Traverse City, Mich., tentative standards for the design, construction, and maintenance of a public water distribution system in Michigan were presented for the consideration of the section. These tentative standards were thereupon referred to a Distribution System Standards Committee for study and recommendations. This 1949–50 committee revised and enlarged the standards and reported them to the 1950 meeting of the section in Detroit. The standards were discussed on the floor and the recommendations made were referred to the 1950–51 committee. This committee has given careful and thorough consideration to the proposed tentative standards and has finally approved the enlarged and revised "Minimum Standards for Design, Construction, and Maintenance of a Public Water Distribution System."

The committee prepared the minimum standards with the understanding that they are for the guidance of experienced water works administrators capable of using discretion in their application. Conditions may be expected to arise for which the minimum standards prescribe no detailed course of action. In such situations the course to be followed must be prescribed by some competent person of discretion. It must be remembered that the standards indicate the minimum requirements and not necessarily the recommended or desirable requirements. Therefore, wherever practicable, designs and methods of construction and maintenance which will result in the greatest functional efficiency, structural security, and sanitary protection of public water distribution systems should be employed.

Distribution System Standards Committee

Robert L. McNamee, *Drury, McNamee and Porter (Chairman)*	1949–51
Edward D. Barrett, *Chicago Bridge and Iron Co.*	1949–51
Raymond J. Faust, *Michigan Dept. of Health*	1949–50
John E. Vogt, *Michigan Dept. of Health*	1950–51
Harold B. Huntley, *Lansing Board of Water & Electric Light Comrs.*	1949–51
Richard King, *University of Michigan*	1949–50

Table of Contents

Minimum Standards for Design, Construction, and Maintenance of a Public Water Distribution System

Sec. 1—General Statement

1.1. *Purposes.* These standards set forth minimum requirements for safe practice in design, construction, and maintenance to insure the functional efficiency, structural security, and sanitary protection of public water distribution systems. These standards presume the delivery into the distribution system of a water supply adequate in quantity and acceptable in quality for human consumption and other domestic purposes. These standards are to be interpreted as indicating the minimum requirements and not necessarily the recommended or desirable requirements.

1.2. *Definitions.* For the purpose of these standards, the use of the word "shall" indicates a mandatory requirement, and the use of the word "should" indicates a recommendation for good practice. As used in these standards, the distribution system includes the complete network of conduits, tanks, reservoirs, booster pumping stations, valves, hydrants, service lines, and other appurtenances used for the transmission of water from its source to its users. As used herein, a water main is a pipeline usually situated in public streets or alleys or rights-of-way for the delivery of water to public and private fire hydrants or to consumers' service lines; a service line is a branch line connected to a main for delivery of water to an individual consumer's premises.

1.3. *Related Laws and Legislation.* Act No. 240 of the Public Acts of 1937, as amended, provides that it shall be unlawful for the state or any of its political subdivisions or any county, city, town, township, village, or school district to engage in the construction of any public work costing more than $2,000 unless the plans, specifications, and estimates have been prepared by and the construction executed under the direct supervision of a registered professional engineer. Act No. 98 of the Public Acts of 1913, as amended, requires that each city, village, private corporation, partnership, or individual operating a water works within the state must file with the State Health Commissioner plans and specifications of the entire system and of any alterations, additions, or improvements thereto; and that, before commencing the construction of any water works system or alteration, addition or improvement thereto, plans and specifications of the same must be submitted to the State Health Commissioner, and a permit for the construction of the same must be secured from the State Health Commissioner.

1.4. *Standard Specifications.* Wherever standard specifications for materials and methods are referred to herein, the specification named, or the latest revision thereof, shall apply, if consistent with other requirements of these standards.

Sec. 2—Functional Requirements

2.1. *General.* The distribution system shall be designed, constructed, and maintained so as to promote and insure the transmission and delivery of water from its source to its users with an economical loss of head, adequate residual pressures, good circulation, and minimum loss of water by leakage.

2.2. *Grid System.* The distribution system, where practicable, should be in

grid form—that is, arranged with parallel pipelines interconnected by lateral pipelines at frequent intervals, so that pressures throughout the system tend to become equalized under varying rates and locations of drafts of water from the system.

2.3. *Sizes of Mains.* The pipe sizes in the distribution system should be such as will maintain an adequate positive pressure of water in all parts of the system at all times. Under normal operating conditions, an adequate pressure should be not less than 35 psi; during periods of peak demand, a minimum adequate pressure should be not less than 20 psi. These objectives can usually be achieved with a grid system of large arterial mains widely spaced in parallel, smaller parallel mains, and interconnecting mains at right angles, supplemented with pressure-stabilizing elevated storage tanks and reservoirs and booster pumping stations. As a general rule, the minimum size main serving a fire hydrant should be not less than 6 in. in diameter; smaller than 4-in. mains should be installed only for temporary service to be later replaced with larger mains, or for secondary parallel mains not for fire service, in wide or paved streets to avoid long or expensive service connections or pavement cuts to the principal main, or for short mains, not for fire service, in courts or cul-de-sacs.

2.4. *Length of Runs.* The lengths of run of small mains should be determined by the local conditions, but in no case should they exceed the limits given in Table 1.

2.5. *Storage Tanks and Reservoirs.* Adequate storage facilities shall be provided to maintain reserve supplies at adequate pressures at strategic points on the distribution system to meet normal and peak demands with additional reserve for fire protection.

2.6. *Valve Locations.* The distribution system should be equipped with a sufficient number of gate valves so located that no single accident, breakage, or repair to the pipe system, exclusive of arterial mains, will necessitate the shutting from service of a length of pipe greater than 500 ft in high-value districts, or greater than 800 ft in other sections, and will not result in shutting down an artery. Valves should be provided at each end of a stream or railroad crossing and on each hydrant branch. The locations of valves should be uniform with respect to street lines insofar as practical.

TABLE 1

Limiting Lengths for Small Mains

Pipe Size *in.*	Length—*ft*	
	No Fire Service	Fire Service
Mains With Dead or Unconnected End		
2	300	inadequate
3	300	inadequate
4	1,300	inadequate
Mains With Both Ends Connected to Larger Main		
2	600	inadequate
3	600	inadequate
4	2,000	600

2.7. *Hydrant Locations.* The distribution system shall be equipped with hydrants so located and spaced as to provide fire protection for the whole population and area served. The locations and spacing in populated communities should provide a hydrant for each partial area, as given in Table 2.

If two fire pumpers can be connected to two steamer outlets on one hydrant, the hydrant may be considered 1½ hydrants. Hydrants should be located to minimize the hazard of damage by traffic.

2.8. *Air Relief, Vacuum Relief, and Blowoff Valves.* At high points in a pipeline not interconnected with a grid system or otherwise provided with means for air or vacuum relief, valves should be provided to release air that may accumulate at the high point, or to admit air to relieve an excessive vacuum that may develop if the line is drained. The point of air intake to a vacuum relief valve should be near the top of the enclosing manhole and above any possibility of submergence by water in the manhole. At extreme low points in pipe systems, such as at stream crossings and at dead ends, a blowoff valve and drain line should be provided for the purpose of draining the line. Blowoff lines should discharge to the ground surface at a point above any possibility of flooding.

TABLE 2

Hydrant Spacing for Populated Communities

Required Fire Flow *gpm*	Required Average Area Per Hydrant *sq ft*
1,000	120,000
2,000	110,000
3,000	100,000
4,000	90,000
5,000	85,000
6,000	80,000
8,000	60,000
10,000	48,000
12,000	40,000

2.9. *Vaults and Manholes.* At each functionally important gate valve and at all gate valves 16 in. and over in diameter, and at each air relief, vacuum relief, and blowoff valve, a vault or manhole should be provided to facilitate the operation and maintenance of such equipment. The vault or manhole should be large enough to serve its purpose and be designed for the traffic loads that it may carry.

Sec. 3—Structural Requirements

3.1. *General.* The distribution system shall be designed, constructed, and maintained to safeguard the structural security of the system both against the functional loads imposed in its operation and against the external loads and the hazards to which it may be subjected.

3.2. *Pipe.* Pipe used in the construction of water distribution systems should be made of cast iron, asbestos cement, reinforced concrete, or steel.

3.2.1. Cast-iron pipe, bell-and-spigot, 3 to 24 in. in diameter, should fulfill the requirements of Federal Specifications WW-P-421 or American Standard specifications ASA A21.2 of class suited to the loading condition involved.

3.2.2. Cast-iron pipe, bolted joint, 2 to 24 in. in diameter, should fulfill the requirements of Federal Specifications WW-P-423.

3.2.3. Cast-iron pipe, 24 in. and larger in diameter should be designed specifically for conditions under which it is to be used.

3.2.4. Cement lining for cast-iron pipe should fulfill the requirements of American Standard specifications ASA A21.4.

3.2.5. Asbestos-cement pipe should fulfill the requirements of Federal Specifications SS-P-351, of class suited to loading.

3.2.6. Reinforced concrete pipe should fulfill the requirements of specifications AWWA C300 for steel cylin-

der type, not prestressed, and AWWA C301 for steel cylinder type, prestressed.

3.2.7. Steel pipe should fulfill the requirements of specifications AWWA C201 for electric fusion welded pipe, 30 in. and over in size, and AWWA C202 for sizes up to but not including 30 in. Coal-tar enamel protective coatings for steel pipe shall conform with AWWA C202 for pipe 30 in. and over in size, and AWWA C203 for pipe smaller than 30 in.

3.2.8 Service pipe used in the construction of service lines connected with water distribution systems should be made of cast iron, asbestos cement, copper, brass, lead, wrought iron, or steel. Cast-iron, copper, brass, lead, wrought-iron, and steel pipe should fulfill the requirements contained in "Collected Standard Specifications for Service Line Materials," an unofficial report of American Water Works Assn. Subcommittee 7S, published with specifications AWWA C800 for threads for underground service line fittings.

3.3. *Appurtenances.* Joint materials, valves, and fire hydrants should be of types and qualities suited to the general standards of the system in which they are installed.

3.3.1. *Joint Materials.* Packing materials used in the joints of bell-and-spigot pipe joints shall be of a type that will not harbor or promote the growth of bacteria, and shall be free of oil, tar, or greasy substances. Molded rubber rings, round or wedge type rubber packing, treated paper packing, and asbestos packing, are generally satisfactory. Jute or oakum should not be used. Jointing compound used in sealing bell-and-spigot pipe joints should be of a type that will promote a watertight seal, while at the same time permitting a slight flexure in the pipeline at the joint, especially under conditions where vibration or settlement may occur. Pig lead poured while melted and calked after cooling, and various prepared mineral compounds poured while melted but not requiring calking are commonly used. Joint materials for other than bell-and-spigot pipe including mechanical joints with rubber gaskets, should be of the type provided by the manufacturer of the pipe and suited to the conditions of installation.

3.3.2. Valves used in the construction of water distribution systems should fulfill the requirements of specifications AWWA C500.

3.3.3. Fire hydrants used in the construction of water distribution systems should fulfill the requirements of specifications AWWA C502.

3.3.4. *Supports.* Pipe should be uniformly supported throughout its length on a prepared sand or soil base with special care to avoid concentrated bearing upon rocks or boulders. Thrust blocks of concrete extended to undisturbed soil should be provided at all plugged ends of crosses, tees, and dead-ends, at the back of hydrants, and on the outside wall of elbows to resist the tendency of water pressure to cause slippage in the joints of plugs, hydrants, and fittings.

3.4. *Location of Mains.* Insofar as practicable, mains in public streets should be laid in a uniform position in the street—that is, on the centerline or a uniform distance to one side of the centerline. This practice will establish a more or less reserved position for this utility, and will facilitate finding valves and other appurtenances at the street surface.

3.5. *Depth of Mains.* All water mains should be laid at sufficient depths to protect the mains against any ex-

ternal static or dynamic loads that may come upon them, and against freezing of water within the mains. In northern latitudes, these requirements will generally be fulfilled by laying mains with a cover over the top of the pipe of not less than 5 ft.

3.6. *Stream Crossings.* Mains to be laid across streams should preferably be placed beneath the stream. Under such conditions the main should be placed far enough below the bottom of the stream bed to protect the main against freezing and against disturbance by currents, ice, floating objects, anchors, dredging, or other forces. Where mains are placed above the stream, the crossing should be made upon bridges, dams, or other elevated structures so that the pipe will not be subject to immersion at any time. Joints in mains at stream crossings should be of a type that will provide a reasonable degree of flexibility while remaining watertight. Valves should be provided at each end of crossings so that the section can be isolated in the event of a break or other emergency. Some means of flushing out crossings should be provided.

3.7. *Railroad Crossings.* Mains to be laid across or along railroads should be designed to withstand the effects of vibration caused by moving trains. Joints should be of a type that will provide a reasonable degree of flexibility while remaining watertight. The placing of such mains in a tunnel or conduit so constructed as to minimize the transmission of vibration to the main is an effective type of installation.

3.8. *Storage Tanks and Reservoirs.* Storage tanks and reservoirs shall be specifically designed and constructed to withstand the functional and external load that may come upon them. Elevated steel tanks, standpipes, and reservoirs should fulfill the requirements of specifications AWWA D100.

3.9. *Installation and Testing.* Cast-iron water mains with lead or compound joints should be installed and tested in accordance with specifications AWWA C600. Mains constructed of other pipe materials or with other than lead or compound joints should be installed in accordance with the above specifications insofar as it is applicable to the pipe material or joint material to be used.

Sec. 4—Sanitary Requirements

4.1. *General.* The distribution system shall be designed, constructed, and maintained to safeguard the quality of the water delivered to the system against contamination from any internal or external sources during its transmission from its source to its users.

4.2. *Construction Sanitation.* Before excavation of the trench is started, all intersecting sewer lines, house sewer connections, and other subsurface drains should be located and provision made to prevent the discharge of wastes from such lines into the trench. If any such lines are disturbed, they should be carefully restored to a tight operating condition. Measures should be taken to prevent defecation and urination in the trench. Suitable sanitary conveniences, connected to sewers where practicable, should be provided for workmen. If sewage does find its way into the trench, it should be removed, and the contaminated area should be disinfected with chlorinated lime. Provisions should be made for the removal of all ground or surface water from trenches, and such water should be prevented from entering water mains being laid. Pipelaying operations should be suspended during rains or whenever the trench cannot be kept

dewatered. A tight stopper or bulkhead should be placed at the open end of a main being laid at all times when pipelaying operations are not in progress.

4.3. *Cross Connections.* Cross connections between the public water distribution system and any other water pipelines shall be permitted only under conditions that fulfill the requirements of the "Michigan Department of Health Regulation for the Protection of Potable Water Supplies in Instances Where Private Supplies Are Employed for Any Purpose Whatsoever," Sec. 1 of which states:

> No physical connection shall be installed or maintained between lines carrying a potable public supply and pipes, pumps, or tanks supplied or possible of being supplied, from any nonpotable source. Where dual supplies are necessary or desired, lines carrying water from the public supply must be protected against backflow of polluted water, by means of an atmosphere gap of not less than 6 in. This may be accomplished by an elevated tank with or without booster pump, or a reservoir from which water is pumped. In every case the inlet line to the tank, carrying potable water, shall discharge at least 6 in. above the maximum possible high-water level of the tank or reservoir as the case may be. Surface water supplies such as streams, lakes, and ponds are not considered potable.

Sections 2 to 7 set forth the regulations under which a service connection between the public water supply system and a potable private supply is permissible.

4.4. *Location Related to Sewers.* Water mains, tanks, and reservoirs should be constructed, insofar as possible, above the elevation of any sanitary sewers that lie within 10 ft laterally of them. Where such a relation cannot be had, extra precautions should be taken to secure absolute and permanent tightness of water-main joints. If practicable, the sewer within the zone of 10 ft laterally from the water main, or 50 ft laterally from a tank or reservoir should be constructed of pipe with watertight joints. Water mains and sewers should not be laid in the same trench.

4.5. *Drainage of Hydrants, Vaults, and Manholes.* Drains from hydrant barrels, vaults, and manholes on water distribution systems shall not be connected to sanitary sewers or storm water drains. Where practicable, hydrant barrels, vaults, and manholes should be drained to the ground surface or to dry wells provided exclusively for that purpose.

4.6. *Storage Tanks and Reservoirs.* All storage tanks and reservoirs shall be of sanitary, watertight construction and shall be made of steel, concrete, or other permanent materials. Tanks and reservoirs shall be located at safe distances from possible sources of contamination. Tanks and reservoirs shall be watertight. Those made of concrete shall be adequately reinforced to prevent the development of cracks, and shall be provided with adequate water stops at all construction joints. Those made of steel shall be thoroughly protected against corrosion by means of suitable protective coatings. All water storage units shall be equipped with watertight covers. Manhole openings in tanks and reservoirs shall be curbed to a height of at least 6 in. above adjoining surfaces. The manhole cover shall be watertight and shall overlap the curbing and extend downward around it for not less than 2 in., and shall be secured in place by a hasp and lock or other equivalent means. All external openings in tanks and reser-

voirs shall be so located and protected that there is no danger of contamination by surface drainage or flood waters. Air vents on storage units shall be constructed of metal tubing or pipe, connected in such a way as to be watertight, and with the open end of the vent screened with 24-mesh brass or bronze screen terminated in a downward direction by means of an elbow or equivalent means, with the lower end of the outlet not less than 12 in. above the roof of the storage unit or less than 24 in. above the established ground elevation. Drains and overflows from tanks and reservoirs should not be connected to sewers, but should discharge to the ground surface at a point above any possibility of flooding. Tanks or reservoirs built in the ground should be located, where practical, so that 50 per cent of the water depth is above the natural ground level and the bottom of the reservoir is above the ground water table. Pressure tanks should be located wholly above ground, or, if below ground level, with one end completely exposed in a pit, drained to discharge to the ground surface at a point above any possibility of flooding.

4.7. *Flushing and Disinfection.* Immediately after construction and before placement in service, every water main should be flushed and disinfected. In the event of subsequent contamination, the main should again be flushed and disinfected. The flushing and disinfection should be carried out in accordance with specifications AWWA C601, a procedure for disinfecting water mains, or "Michigan Department of Health Regulations for Cleaning and Disinfection of Water Mains." New storage tanks or reservoirs, or those which may have been contaminated or subjected to the possibility of contamination during cleaning, alterations, painting, or repairing operations should be disinfected before being placed in service. The underside of the roof of the tank or reservoir should be washed with clean water. The walls and floor of the tank or reservoir should be thoroughly mopped with a solution having a free chlorine concentration of at least 100 ppm. The walls and floor should then be rinsed by washing with a stream of water, which should then be wasted. The tank or reservoir should then be filled with water dosed with a chlorine solution of such strength as will produce a residual chlorine content of at least 1 ppm at the end of a 3-hr period. After such a residual has been secured at the end of a 3-hr period, the reservoir and the water in it may be restored to service.

Sec. 5—Maintenance

5.1. *General.* All elements of the system should be operated and maintained in such manner as will insure its functional efficiency, structural security, and sanitary protection.

5.2. *Personnel.* All personnel responsible for operation and maintenance of the system shall be qualified by training and experience for their respective duties. They shall be informed fully of the functional, structural, and sanitary characteristics and requirements of the elements of the system with which their duties are related.

5.3. *Equipment.* Equipment required for operation and maintenance of the system shall be suited to its purposes and shall be kept ready for service.

5.4. *Maps and Records.* Complete and up-to-date maps of the system, drawn to suitable scale, shall be maintained at conveniently available locations throughout the system. The locations of pipelines, tanks, reservoirs,

booster pumping stations, valves, hydrants, service lines, and other appurtenances shall be accurately recorded thereon. The direction of rotation of valve stems and the number of turns to close should be noted. Detailed engineering drawings and descriptive records of operating equipment and appurtenant pipelines and electrical equipment shall be preserved in some accessible place.

Editors Note: The "Minimum Standards for Design, Construction, and Maintenance of a Public Water Distribution System" constitute a voluntary agreement between the AWWA Michigan Section and the Michigan Dept. of Health, and are not official AWWA standards. Standards of minimum requirements were also adopted by the AWWA California Section in 1948 and were published in the January 1949 Journal. The Michigan standards were prepared and subsequently adopted on Sept. 20, 1951 by the Michigan Section at the request of the Michigan Dept. of Health. The need for standards was emphasized to the department by the frequent requests for construction permits to build small water main extensions supposedly costing less than $2,000 (see Sec. 1.3 Related Laws and Legislation). Most of the plans for these extensions were not designed by registered professional engineers. Many of the designs violated sound engineering principles—especially in pipe sizes, location and number of valves, pipe materials, fire flows, and provisions for future requirements. The department could issue construction permits only after new sets of plans showing properly designed extensions were received—usually after long delays.

Ground-Wire Attachments to Water Pipes

By R. C. Kennedy

A paper presented on Oct. 26, 1951, at the California Section Meeting, San Francisco, by R. C. Kennedy, Chief Engr., East Bay Municipal Utility Dist., Oakland, Calif.

SECTION 24 of the "Tentative Regulations Governing Water Service," which were approved by the California Section AWWA on Oct. 27, 1950, states (1):

> All individuals or business organizations are forbidden to attach any ground wire or wires to any plumbing which is or may be connected to a service connection or main belonging to the utility; the utility will hold the customer liable for any damage to its property occasioned by such ground wire attachments.

The author prepared this portion of the regulations in accordance with established AWWA policy (2) and the regulations of the East Bay Municipal Utility Dist., which contain a similar prohibition against ground-wire connections, although the rule has never been actively enforced. Therefore, the numerous objections which were promptly registered came as a surprise. Since that time, the matter has been further investigated and the author is now even more thoroughly convinced that the prohibition is essential in the regulations of every water supply utility.

The difficulty with Section 24 stems largely from the fact that it is directly contrary to the requirements of the *National Electrical Code,* the Electrical Safety Orders of the state of California, Division of Industrial Safety, and probably most city electrical codes. Section 2612 of the *National Electrical Code* states:

> System or common grounding conductors shall be attached to a water piping system on the street side of the water meter or on a cold water pipe of adequate current-carrying capacity as near as practicable to the water service entrance to the building. . . . If the point of attachment is not on the street side of the water meter, the water piping system shall be made electrically continuous by bonding together all parts between the attachment and the street side of the water meter or the pipe entrance which are liable to become disconnected, as at meters, valves, and service unions. . . .

State and city codes also include this requirement by reference to the *National Electrical Code.*

Damage to Water Pipes

There is an obvious and basic objection to an order which requires a property owner to use and make attachments to the utility's pipelines for a purpose foreign to their normal function, and without the utility's permission. From a practical point of view, however, there are two main questions, the first of which is: Does grounding of alternating electric circuits to water pipes cause electrolytic or other damage to the pipes?

The usual electric service to a customer's premises begins with a transformer at the street line by which high voltage is reduced to 220 and 110 v. Two wires extend into the building, one of which is "live." The other, or neutral wire, is grounded to the cold water piping system. If 220 v is required for operating such equipment as a cooking range, three wires are brought in—the two transformer secondary wires plus the grounded center tap of the transformer. The two transformer secondary wires comprise a single 220-v circuit when taken together; in addition, each of them separately, in conjunction with the grounded center wire, provides one 110-v circuit. The gas main, incidentally, is commonly insulated from the house lines by a special coupling to avoid damage to the gas meter from electric currents passing through it.

When a switch is thrown, current flows through the live wire to the equipment, and, if only one service is supplied from the transformer, the current flows back to the transformer through the neutral wire. Under such circumstances, the grounding water main carries no current. If more than one service is taken from the transformer, however—which is usually the situation in heavily populated areas—only a part of the return current flows as described through the neutral conductor on the same property; the remainder travels through the water service piping and main to the neighbor's neutral conductor and then back to the transformer. The amount of the current taking this latter path varies from 0 to 100 per cent of the load, with an average of approximately 20 per cent, depending upon the relative resistance of the two parallel paths. The neutral side of the transformer secondary is also grounded by means of a driven rod at the transformer pole. Some current may therefore be returned by this path, but it would normally be small unless the other grounding means is defective.

In an effort to determine the damage, if any, caused by alternating currents passing over the water pipes, the American Research Committee on Grounding, sponsored by AWWA and the Edison Electric Inst. and having representation from eighteen other national organizations, made an extensive study of the subject and reported (3):

1. Effective grounding of electrical circuits is essential to the safety of persons and property.
2. Water pipes are the most efficient grounding medium, and regardless of what other grounding device might be used, incidental metallic contacts usually cause the water piping to become the actual ground.
3. The existence of water facilities throughout most buildings results in many contacts with people and makes it necessary that equipment be grounded to the piping because, if other means were used, the potential difference that might exist between the piping and the otherwise grounded equipment could result in shock.
4. The voltage on the water lines used for grounding purposes appears to be approximately 3 to 4 v.
5. In spite of the large current carried, no damage to piping or to the water within it resulting from alternating current grounding was discovered by the committee, although many suspected cases were carefully examined.

The committee's study was apparently thorough and unprejudiced, and

the author feels that its findings must be accepted until other evidence is produced. Thus it may be said that grounding of alternating current equipment to water pipes does not cause damage to them.

Legal Liability

The second question—Can grounding to water pipes involve the utility in a legal liability?—is not so easily answered. Consultation with attorneys has given the author some insight into the problem.

The *National Electric Code* states: "circuits are grounded for the purpose of limiting the voltage upon the circuit which might otherwise occur through exposure to lightning or other voltages higher than that for which the circuit is designed; or to limit the maximum potential to ground due to normal voltage."

Three common situations may create dangerous conditions on the customer's premises through loss of the protective electrical ground:

1. The use of nonmetallic water mains such as asbestos-cement
2. The use of nonmetallic service lines such as the numerous plastics now becoming popular
3. The removal of the water meter, even temporarily, unless a bonding jumper is substituted. (If such jumpers are not installed at meters, the code is being violated by the property owner.)

It is recognized in the *National Electric Code* that 10 ft of buried pipe in normal earth constitutes a favorable electrical ground. In dry ground, however, such as some sandy soil or the soil under a paved driveway, 10 ft of buried pipe furnishes little effective protection. Frequently the meter is actually within the basement of apartment houses or commercial buildings, thus affording no ground whatever inside the meter location. The occupants of these buildings are in danger if reliance is placed on water pipes for the grounding medium, unless the service line and main are metallic and continuous.

Every utility engineer is fully aware that the water piping is used for electrical grounding, and that it constitutes an important safeguard against personal injury and property loss. Under the law he is endowed with "superior knowledge" of these facts. It is impossible for him to avoid the use of piping facilities for this purpose, and the law governing the installation of electrical equipment compels the electrical contractor to use them. Can he then take away the protection afforded by his facilities without liability for damage resulting from this act? It will be interesting to learn the decision of the court in the first case over this question, but meanwhile there are three things which the utility engineer can do to protect his organization:

1. Prohibit the use of water piping for grounding purposes, even though the rule is not actively enforced, and give each new customer a copy of the regulations so that he will have full opportunity to know the conditions under which service is granted.
2. Notify all licensed electrical contractors that the water system is not provided for grounding purposes, and that pipelines will not necessarily be permanently suitable for this purpose.
3. Notify the customer in writing whenever the normal continuous metallic ground connection afforded by the water pipe is interrupted.

Other Solutions

There is another possible approach to this problem which deserves careful consideration. Possibly water utilities should recognize the dual use of their facilities, and provide suitable grounding at all customers' premises as a part of the service. If this decision is reached, all meters should be provided with properly designed bonding jumpers, and nonmetallic service lines or mains should be used only where sufficient ground contact is assured by the remaining metallic piping not so isolated. Also, the types of pipe joint and the conductivity of the jointing materials would have to be considered. This alternative becomes another legal question, however, on which an attorney's views are required.

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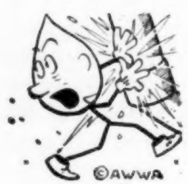

Notes on Modern Swedish Water Supply and Treatment Practices

By Amos J. Alter

A contribution by Amos J. Alter, Director, Div. of Sanitation & Engineering, Alaska Dept. of Health, Juneau, Alaska.

MANY water supply and water treatment practices now used in Sweden are of general interest—some because of the lack of similar practice in continental United States, others because of a somewhat different approach to problems well known to the American water supply and treatment profession. Modified design for rapid sand filters, development of artificial ground water supplies, and the use of a combination of activated calcium and magnesium carbonate with magnesium hydroxide are but a few of the interesting Swedish practices.

Sweden, a country of approximately the same size and population as California, enjoys a climate and has a terrain quite similar to the British Columbia coast of Canada. North Sweden, being somewhat removed from the influence of coastal waters, experiences a subarctic climate. Manufacture of iron and steel, electrical machinery, porcelain, glass, and forest products, as well as the primarily agricultural economy of the country, has presented extensive water supply needs for both industrial and domestic purposes. Stockholm, the largest city, with a population of approximately 1,000,000 in the city and environs, has made many advances in providing safe water supply since the establishment of the water works department in 1861. As in Stockholm, the utilities in most communities of Sweden are under public ownership. Although public water supplies are large in number, most of them are small in size. Only two or three of the largest are equipped to carry on investigative and developmental work.

Physical, chemical, and bacteriological standards for domestic water supply are fairly high throughout the country. The greatest part of community water supply in Sweden is provided from surface sources. A considerable number of communities all over the country, however, are supplied with ground water. Most of the surface waters are relatively soft, fairly clear, and highly colored because of a high content of organic matter.

Artificial Ground Water Supplies

Since the development of the first ground water recharging plant in Sweden by J. G. Richert in 1897, the use of artificial ground water has increased gradually until now approximately 10 per cent of the urban population is served from such sources. Artificial ground water recharge methods were adopted at Sala in 1903, Orebro in 1918, Luleå in 1932, Eksjö in 1936, Hälsingborg and Katrineholm in 1937, and the communities of Karlskoga, Kristinehamn, and Eskilstuna in 1949.

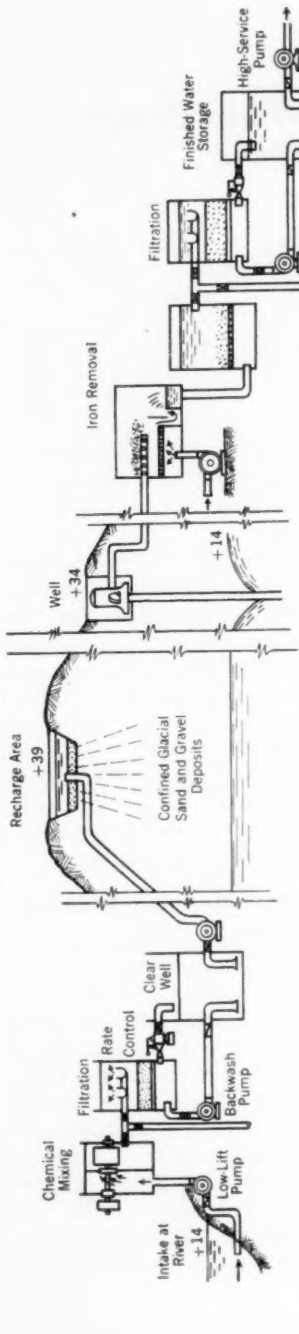

Fig. 1. Arrangement of Artificial Ground Water Recharge Facilities at Eskilstuna, Sweden

Facilities for artificial recharge are to be completed at Malmö, Västerås, Karlstad, and some other towns within the next two or three years. Scottish experiments in 1810 and French experiments a few years later merely utilized filter galleries which provided natural filtration. Direct application of surface water to a point on the surface of the soil from which it is allowed to percolate into the ground water, is typical of present-day Swedish practice. As clogging of filter beds or the surface of the recharge area with algae and organic material has often presented problems, the need for effective pretreatment of surface water prior to discharge into the natural aquifer has been recognized.

A diagram of the ground water recharge facilities for the city of Eskilstuna is presented in Fig. 1. In this community, the surface water contained much organic material. Recharge facilities provide for its continuous removal prior to application of water to the recharge area. Slightly turbid water would otherwise gradually penetrate and clog pores of the soil, thus destroying any purifying capacity of the aquifer.

Glaciation in Scandinavian countries has left unsorted morainal deposits, partly covered by clays. Such deposits are somewhat impermeable and discontinuous, thus being of little value as a source for a large ground water supply. The appearance of gravel eskers as a result of the glacial action has, however, in many places, left long and narrow aquifers consisting principally of sand and gravel. Although storage capacity of these aquifers is quite low, they make an excellent natural reservoir for use in artificial recharge of ground water. The recharge area at Eskilstuna is over an esker.

Earlier artificial recharge plants, which did not provide for preparation of the water prior to recharge, suffered experiences such as that at Sala, where, although a distance of 500 ft was left between recharge basin and pumping well, highly colored and turbid water completely penetrated the natural sand layer and well water began to increase in color in a period of 35 years.

It has been found that the rate of recharge of water to an esker must be carefully regulated and that it is preferable that flow be continuous during winter months to prevent freezing. According to consulting engineer Victor Jansa of Stockholm, percolation of water through the air-filled layers of gravel produces an action like that of a trickling filter, and decrease in oxygen content in the recharge water indicates that an oxidizing process occurs. Jansa proposes that further aeration be provided by injection of air into the ground at some depth below the infiltration area. The temperature equalizing effect, which passage through the esker exerts upon the water is of considerable importance. The water is somewhat warmed in winter and cooled in summer. The ideal storage time for recharge water is assumed to be six months. Thus, with optimum storage, water which is stored in the winter would be taken from the well supply during summer. Artificial recharge methods have been used successfully in freshening a well which produced fresh water at installation, but later produced salt water. Natural recharge methods for lowering the salinity of ground water are commonly used, particularly in countries like Holland where salt water intrusion presents a continuing problem.

It is claimed that Swedish recharge methods:

1. Make coagulation and settling unnecessary.
2. Through biological action in a natural aquifer, afford the most suitable means for taste and odor control.
3. Make surface waters more palatable by temperature adjustment.

Treatment Plant Modifications

Filter underdrains in some modern rapid sand filtration plants in Sweden are constructed by using a perforated screen or strainer made of noncorrosive metal placed on a false bottom. It is believed that use of such a perforated bottom provides much better distribution throughout the filter. Slightly larger sand particles than now used in the United States or previously used in Sweden are being employed in new filters. Grading of sand in new type filters of the leading water treatment plants (among them Stockholm) ranges from 0.9 to 1.5 mm. More extensive use is also being made of surface washing equipment to supplement the reverse flow washing. Surface wash is with water jetted from a fixed pipe system above the filter surface, usually at a rate of approximately 3 to 4 gpm per sq ft, whereas the reverse flow wash water rate is approximately four to six times the flow rate for the surface wash. Use of slow sand filters as a finishing process for taste and odor control is receiving considerable attention at many water treatment plants in Sweden. Usually, however, modern rapid sand filters are not followed by slow sand filters.

It is reported that the customary "schmutzdecke," which accumulates on slow sand filters, is not rapidly formed when such filters are used only for a finishing treatment to the effluent from a rapid sand filter plant. It has also been found that the "schmutzdecke"

is not essential in the use of the slow filter for taste and odor removal. This action of the slow sand filter is attributed to bacterial activity influenced by chemical oxidation of organic compounds dissolved in the water. The consumption of oxygen is easily demonstrable in filters used as finishing filters. Alkalizing the water passing through the slow sand filter to a pH of approximately 8.5 (inhibitive to protozoa and abundant algae life) makes taste and odor control more effective than is experienced when water with a lower pH is passed through the filter. Small amounts of nitrogen and phosphorous added to the water prior to slow sand filtration also appear to improve functioning of the filter for finishing treatment. A ripening period is required, however, if the slow filter is to do an effective job.

Some Swedish authorities report that disinfection of the water by chlorination or similar means, prior to slow sand filtration, makes operation of the filter ineffective. Further investigation of this point is necessary. A considerable amount of the experimentation on the use of the slow sand filter for taste and odor control has been done at the Lovö Water Purification Plant in Stockholm.

"Elektromagno," a material of which both the manufacture and use have been patented, is now receiving some attention as a chemical for use in municipal water treatment. Although no Swedish plant of importance has adopted the material, its application to water treatment problems in small public supplies as well as in swimming pools is being carefully followed.

The material consists of dolomite heated in a special process. First developed for use in corrosion control, it also appears to be an effective agent for removal of iron and manganese, especially for use in small installations. "Elektromagno" is produced as a granular material and is used as a filtering medium both in pressure and gravity type filters. Grain sizes range from 1 to 4 mm. The grains are of irregular shape and are expended through chemical combination in the treatment process, as well as through mechanical wear and losses in wash water. Theoretically, 1 g of carbon dioxide combines with 1.2 g of "Elektromagno." In general practice, however, 1 g of carbon dioxide may be expected to combine with or use 1.5 g of "Elektromagno." Filtering rates through the specially prepared filter range from 3 to 6 gpm per sq ft for pressure filters, and from 2 to 4 gpm per sq ft for gravity filters. Backwash rates are reported to be approximately 12 gpm per sq ft.

Use of "Elektromagno" for corrosion control for either soft or hard waters depends upon formation of a carbonate film as the result of pH adjustment or calcium carbonate balance. Adjustment of pH and calcium carbonate balance are said to be automatic under operating conditions. With the material, pretreatment by free residual chlorination during a long contact period makes iron removal feasible if it occurs as an organic salt in combination with humic acids. The degree of oxidation is dependent upon pH, a high pH facilitating oxidation. Thus, through prechlorination and pH adjustment, removal of iron from very soft waters may be accomplished. Reportedly, increased velocity, made possible through use of the irregular-shaped grains which exhibit a greater surface area, results in instantaneous formation of a heavy floc directly at the point where the surface activity oc-

curs. Since most activity occurs at the surface of the filter, the greatest part of the precipitant is caught in the upper portion of the filter, thus leaving the filter in a condition for rapid and easy washing. "Elektromagno" has also been successfully used for the removal of manganese.

Summary

Many interesting water supply and treatment practices are being used in Sweden. Some of these practices may be suitable for more extensive use in the United States if geological, physical, and economic considerations justify their adoption. Treatment of surface waters before utilizing them in recharging confined ground water aquifers; use of perforated underdrains for rapid sand filters; increased sand size and surface washing with water for rapid sand filters; and the use of chemically active filters in corrosion control and in iron and manganese removal—all are receiving much consideration and wide use in Sweden at the present time.

Acknowledgment

The author gratefully acknowledges the willing assistance given to him by so many persons in Sweden both during and after his visit to that country. In particular, Victor Jansa, Nils K. G. Westberg, Harry L. Schein, and Adam Goldschmidt, all of Stockholm, offered invaluable help.

Rates and Rate Structures for Water Utilities in Pennsylvania

By W. James MacIntosh

A paper presented on Sept. 21, 1951, at the Pennsylvania Section Meeting, Philadelphia, Pa., by W. James MacIntosh, Counselor at Law, Morgan, Lewis, and Bockius, Philadelphia, Pa.

THE supply of water to the public is a field in which public ownership and operation have been predominant for many years. There still remains in Pennsylvania, however, a large number of privately owned water utilities. The principles developed by the Public Utility Commission and the courts to govern rates and service by private companies to a very large extent also control the rates and rate structures of publicly owned systems. This paper reviews some of the more significant differences between publicly and privately owned water utilities, including methods of financing, source of revenue, annual revenue requirements, and government regulation.

Methods of Financing

Among the most significant differences in the financing methods available to the two types of ownership are: [1] the ability of certain types of publicly owned water utilities to assess adjacent property; [2] the existence of equity capital—that is, common and preferred stock—in privately owned utilities; [3] the use of revenue and general obligation bonds by public utilities as opposed to the use of mortgage bonds and notes by private companies; and [4] the availability of federal aid to certain publicly owned water utilities.

Assessment of Adjacent Property

One of the principal sources of capital available to certain types of publicly owned utilities arises from the authority to assess the cost of certain capital additions against the adjacent property which is benefited. By comparison, the private utility has no similar means of financing open to it except to the limited extent by which it is able to require customer contributions generally under some form of deposit and refund agreement. The existence of this difference undoubtedly has an effect upon the willingness of each type utility to extend its service into new and undeveloped areas. The privately owned utility cannot make such extensions unless the revenue from the additional customers secured is sufficient to meet the added expenses involved and to provide a return upon the needed capital investment. At present, capital costs are so high that the private company can seldom afford to make such additions without substantial contributions from the customers. Those publicly owned systems which have the power to assess adjacent property do not have the same

problem because a major portion of the money required to meet the capital expenditures can be provided through assessment. Moreover, the assessment can be levied upon all adjacent property regardless of actual use so that the initial capital outlay is more equitably distributed than that of a private company which can only secure contributions from the immediately prospective users.

The power to make assessments has a definite effect upon the rates charged by the utility. It is possible, generally, to establish lower rates when the cost of capital expenditures is made through assessments rather than through other sources of capital upon which interest or return must be paid. When customer contributions are deducted from the rate base of private utilities, the contributions also have a tendency to reduce the rates which are charged. Under present methods for securing such contributions, however—that is, agreements providing for deposit of funds with refund over a period of time, as new customers are added—the customer's contribution remains a liability of the private water company and, therefore, should not be deducted from its rate base so long as the potential liability to refund continues.

Equity Capital

Privately owned water utilities are always financed, at least in part, by equity capital, whereas this source is not available to publicly owned utilities. This existence of equity capital is reflected in a different attitude toward the return which the private company must earn and, therefore, has a definite effect upon the rates which are charged by it. The private company must earn a sum over and above its expenses sufficient not only to pay the interest on its funded debt—its bonds and notes—but also to pay dividends upon its common and preferred stock. Its failure to earn such a return will seriously affect the marketability of its securities and will impair its ability to secure additional capital when capital expenditures are required. The publicly owned utility, of course, does not have this problem and as long as it is able to meet its operating expenses and to pay the interest and amortization requirements on its debt, it is under no obligation to secure a return over and above these amounts. To the extent that it does secure a return above costs, however, its ability to borrow additional money at low interest rates is improved.

Bonds

Although the debt financing of the privately owned utility is done almost exclusively through mortgage bonds and to a lesser extent through short-term bank borrowing, the publicly owned utility is able to finance through general obligation bonds, which place the taxing power of the municipality behind the debt, and through revenue bonds, which provide for payment of interest and principal out of the revenues of the particular project being financed. These different methods of financing bonded indebtedness affect the level of rates and charges of the two types of utilities in two ways: [1] the publicly owned utility tends to benefit by lower interest requirements resulting primarily from the tax-exempt status of municipal obligations, so that the needed total annual revenue is reduced; and [2] the publicly owned utility is generally required to amortize the principal of its funded debt, whereas

private companies seldom do this to any significant extent. The latter fact may tend to increase the annual revenue requirements of the public company so as to offset partially the additional return which the private company must earn to support its equity capital. Of course, less additional revenue will be required if depreciation reserves are used for this purpose.

Federal Aid

Not infrequently the publicly owned water utility is able to secure loans and grants from the federal government to extend or improve its system. This source of funds is seldom open to private companies, except for limited assistance they may receive for extension of facilities during wartime to meet increased demands created by enlarged government and industrial facilities.

Sources of Revenue

The principal sources of revenue for both publicly and privately owned utilities are the rates and charges which are made against the water users. Frequently, however, publicly owned utilities have also been permitted to make an annual charge against property regardless of whether the particular property actually uses the service or not. Such a charge is levied on the theory that even nonusers derive considerable benefit from the existence of the water system and that they should be required to pay for that benefit. In the report of the Joint Committee on Rates and Rate Structures (1), it was recognized that such a charge is proper and that, whenever possible, making a charge to nonusers should be continued and extended.

The authority of publicly owned utilities to make such a charge is not clear in all jurisdictions. The privately owned water utilities, however, clearly have no such authority, and some members of the Joint Committee seriously doubted whether such a right should ever be conferred upon private utilities, regardless of the extent to which they may be regulated by public utility commissions. It is clear, however, that a charge to nonusers for the benefit which they receive from a utility is fair, and that such charges, at least by publicly owned companies, should be encouraged.

Both publicly and privately owned water utilities have a source of revenue in the charges for public fire protection. A private company usually makes the charge against the municipality which, in turn, collects it from the citizens of the community through its general taxes. Inasmuch as the existence of this service is of benefit to the entire community, it would seem reasonable that its cost should be returned to the utility by all of the citizens. Many publicly owned utilities make no charge for fire protection service, however. If they fail to make this charge, the actual customers of the company may be required to pay a greater percentage of the revenue requirements of the water system than is fairly allocable to them, unless the difference is adjusted by means of municipal contributions to the utility out of funds collected from other sources.

Annual Revenue Requirements

The most significant distinctions between public and private water utilities are in the annual revenue requirements.

Operation and Maintenance Expense

With minor exceptions, the operation and maintenance expense of a

utility should be approximately the same regardless of the type of ownership. Many publicly owned utilities do not maintain separate accounts, however, and engineering, billing, and collecting services are frequently provided by the municipality itself without careful allocation of their cost to the water system. This practice shifts the burden unfairly upon the taxpayers at large unless it is offset by payments from the water system into the general fund or other equivalent benefits.

Depreciation

Inasmuch as the property of a water utility will depreciate at the same rate regardless of the type of ownership, annual depreciation should be accounted for by both publicly and privately owned utilities. Privately owned utilities are uniformly required to account for depreciation, but publicly owned utilities are not. Frequently depreciation is forgotton entirely or is covered only to a limited extent by the amortization requirements for paying off the bonded indebtedness of the utility.

Taxes

The exemption of publicly owned water utilities from federal income taxes and from most state income taxes is the most significant difference between publicly and privately owned water utilities. With corporate income tax rates above the 50 per cent level, the private company's rates must produce net earnings before taxes in excess of $2.00 in order to provide each $1.00 of net earnings per share of equity capital. As profits are required to maintain the marketability of its securities and its ability to secure additional capital, the requirement of revenue to meet taxes is an important one for the private company. The publicly owned utility, which can be operated at a profit without this burden, should be in a position to maintain a lower level of rates or a higher level of capital improvements financed through income.

Debt

The interest-on-debt requirements of both private and publicly owned utilities are substantially the same except that the publicly owned utility can generally secure lower interest rates than those available to the private companies. The private utility does not retire its outstanding debt except to a limited extent, but, rather, refunds it from time to time as it becomes necessary or advisable. The publicly owned utility, on the other hand, usually makes substantial annual disbursements for this purpose. If the debt retirement requirements are equal to or less than the annual allowances for depreciation, however, it is possible for the publicly owned utility to use its depreciation funds for this purpose. If the requirements are greater than the annual depreciation allowance, the utility will require additional funds which must be secured through its rates and charges. Funds secured through depreciation accruals are also available for property additions, however, and if these funds are used to retire existing debt, it may ultimately be necessary to incur additional indebtedness to meet necessary improvement costs.

In addition to the above requirements, the private utility must earn a return sufficient to maintain the value of its equity securities and to attract new capital when required for exten-

sions and improvements. The publicly owned utility, on the other hand, has no such requirement and, except to the extent that additional revenues may affect the interest rates at which it can borrow, there is no need for it to earn a return above the revenue requirements heretofore mentioned. When publicly owned utilities do earn additional revenues, they frequently pay the excess into the general fund of the municipality, probably on the theory that it constitutes a return to the taxpayers on their investment. The wisdom of following such a practice, as opposed to reducing rates in order to eliminate the excess, is controversial, particularly if the plant has been financed by earnings of the water system rather than by proceeds of general taxes.

Government Regulation

In the regulation of rates of publicly and privately owned water utilities in Pennsylvania, differences exist in both the principles applied and the method of enforcement of the principles.

The most comprehensive discussion of the principles which govern the rates of publicly owned water utilities in Pennsylvania is found in the decision of the Supreme Court in *Shirk* v. *Lancaster City*.* In that case the court indicated that a municipal water system should establish rates sufficient to cover operating expenses and contingencies, a depreciation charge, and a fair return on the present value of the plant. The court indicated that fair value should be determined in the same manner as that of a private corporation, that debt service charges should be compensated from the return allowed, and that any profit over and above this amount could be used to meet general governmental expenses. The court did recognize, however, that at least in theory, a municipally owned plant should provide service without a profit as that term is normally understood in the utility field, and that profits should not be so large as to produce unreasonable discrimination or preference in the tax burdens.

Although the Lancaster City decision would seem to indicate that the principles governing rates for publicly owned water utilities were identical with those governing the private companies, it does point out one significant difference. A municipally owned system, which furnishes service to inhabitants of the suburbs, it was observed, must segregate its property and expenses between that required for service to the suburbs and that for service to the city. In later decisions, it has even been recognized that a municipally owned plant may treat customers served outside its limits differently by demanding a profit from them while foregoing all or a portion of the profit from those within the city limits.†

Since the Lancaster City decision, one further significant difference between publicly and privately owned water systems has developed on the basis of the principle established in *State College Borough Authority* v. *Pennsylvania Public Utility Commission*.‡ This decision held that if operating costs plus a reasonable return on the proper rate base did not provide adequate funds to meet bond obligations,

* 313 Pa. 158 (1933).

† *Ambridge Borough* v. *Pennsylvania Public Utility Commission*, 137 Pa. Super. 50 (1939), and *Altoona* v. *Pennsylvania Public Utility Commission*, 168 Pa. Super. 251 (1951).

‡ 152 Pa. Super. 363 (1943).

an additional allowance should be made for that purpose.

For the enforcement of these principles of rate regulation, a privately owned company is under the jurisdiction of the Public Utility Commission, whereas a publicly owned utility, except for service outside the municipality is subject to the supervision of the courts and to such influence as the voters may have. The chief significance of this difference lies in the fact that the private company generally has the burden of proof in establishing the fairness of its rates or the adequacy of its service whereas the publicly owned utility is required to uphold its rates and service only when an action is brought by a particular citizen or group. Even then, the burden of proof rests with the complaining party. To some extent this difference may make it easier for the municipality to increase its rates when additional revenue is required, although the pressure of the voters, together with the supervision of the courts, provides an effective method of regulation and control when the issues involved are of major importance to any substantial consumer or group of consumers.

Recent Legislation

A number of bills introduced in the 1951 session of the Pennsylvania Legislature were directly concerned with the water works field. Five of these represented an effort to use water utilities as a source of information, service, or enforcement for municipal sewage works, and were undoubtedly prompted by the rapid development in municipal sewage treatment and the need for adequate and sound methods of charging for the service and of enforcing that charge, in order to permit the financing of improvements on reasonable terms. Two of these bills, which were finally enacted, required water utilities serving first and second class townships to furnish a list of water meter readings or flat-rate computations as a basis for ascertaining township sewer service charges. The other three bills, which dealt with municipal authorities and municipalities generally, contained similar provisions with respect to furnishing information but, in addition, they authorized and required water utilities to shut off water service for nonpayment of sewer service charges and authorized the water utility to provide billing and collecting service. These bills provided for compensation to the water utility for these added services but unlike the first two they did not provide for reimbursement of the utilities' reasonable expenses in furnishing information. One of these bills, which dealt with sewer authorities of second class counties and first class cities, became law, but only after a House of Representatives amendment exempting privately owned water utilities from its requirements was adopted. Further efforts to secure this type of legislation may be expected in future sessions, but so long as its application is limited to communities in which two separate municipal authorities are involved in providing the two types of service, it will represent no serious departure from normal practice.

Further legislation which faced no serious obstacle was enacted to provide municipal authorities with the power of eminent domain over small minority stock interests in properties they were acquiring. Another bill which enabled municipalities to require water utilities to introduce fluo-

rides into their supplies, with no provision for reimbursement of the utility for this service, did run into opposition and ultimately failed to pass the legislature.

Conclusion

This review of the differences between publicly and privately owned utilities has necessarily been brief. The author has endeavored merely to emphasize those differences which have particular significance in the determination of fair and proper rates and charges for service. Although the differences in controlling legal principles are relatively slight, wide variations in practice exist. To a large extent the recent studies of the Joint Committee on Rates and Rate Structures have been designed to further the general understanding of the controlling principles and the reasons which lie behind them so that the discrepancies in actual practice may be gradually reduced or eliminated.

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Suburban Main Extension Policies

By W. Victor Weir

A paper presented on Feb. 13, 1952, at the Indiana Section Meeting, Indianapolis, by W. Victor Weir, President & General Mgr., St. Louis County Water Co., St. Louis, Mo.

WATER main extension policy has always been a water utility problem and one which has grown progressively more critical. Inflation and its resultant increased cost of extensions and plants grinds the water utility on one hand, while the unrealistic belief that water rates shouldn't be increased grinds on the other. Either of these forces—unrealistic rates or high cost mains installed under an antiquated extension policy—can get a utility into financial trouble.

The automobile has changed the complexion of the water main extension problem. No longer is community development the regular enlargement of an urban area. Just as long as a real estate development is within commuting distance by auto, a development may be successful. Many calls for water main extensions are made for sporadic developments in the satellite areas around a city. The odds are that these extensions may not even be within the city limits, and may be in areas which the city will not annex for years to come.

Financing Extensions

The first problem to be solved in considering main extension policy is: Shall new customers requiring extensions be subsidized at the expense of the old customers? Many people do not understand the role "old customers" play in extension rule policy. They think the question of financing an extension is a matter entirely between the water department or the water company and the person or real estate firm wanting the extension. The usual reaction is that extension rules should be liberal, and that any harm to the utility is inconsequential.

The utility may not suffer if annual rate increases are made, but annual rate cases are undesirable from every standpoint. The ability of a utility to finance improvements of plant and primary main enlargements may, however, be injured if overly large investments are made in distribution main extensions, service pipes, and meters.

Actually, the water utility is simply a middle man in the problem of main extensions. If extension terms are overly favorable to new customers, they have to be unfavorable to old customers. The water rates to old customers will have to be increased, not only because of any increased expenses for serving them, but also because their water rates must include the subsidy given to new customers.

Many utilities have considered changing extension rules written ten or fifteen years ago, but have met with comparatively little success. There are several reasons:

1. The old customers, or the persons who should represent them, don't realize the injury done to them through higher water rates caused by overly liberal extension rules. The old customers, as a group, are not vocal.

2. The real estate people needing extensions are a vocal group and are usually well organized. Pressure groups usually get things their way, with the result that antiquated extension policies are allowed to remain.

3. The feeling that community growth should be encouraged may be so strong that all the costs of such promotion may not be known or considered. Few people realize that their water rates have been or may be increased simply because the community has gotten larger. They mistakenly believe costs should tend to be lower because more people are being served.

4. Often the utility managements, or the persons who write extension rules, have not realized the impact unrealistic extension rules may have upon the water supply business. They have an idea that the extension rules may be inequitable, but they don't know what to do about them.

Municipal and Private Utilities

The AWWA Committee on Water Main Extension Policy has endeavored to find a common ground between municipal and private water utilities which would permit making water main extension policies similar for each type of operation. It has failed because of essential differences: [1] the municipal utility serves at cost, and "cost" may include large or no capital charges depending upon whether the system is being paid for, or has been paid for, whereas the private utility must always include a return upon the investment made, but cannot get a return of capital in the water rates in excess of reasonable depreciation charges; and [2] within its city limits the municipal water utility is a quasi-governmental operation, whereas the private utility is strictly and wholly proprietary.

When a municipal utility operates outside its city limits, however, its situation parallels that of a private utility; it is there as a propietary agency. Therefore, when suburban water main extension policy is considered, there should be little difference between municipal and private operation. Each should receive a profit—that is, a return upon the investment made to serve suburban customers—and the formulas governing suburban extensions should be similar.

Also, when a municipal utility, as well as a private utility, starts serving a suburban area outside its city limits, it is generally under obligations to serve, and to expand service, in the suburban area. On Jan. 12, 1951, the Pennsylvania Superior Court required the city of Altoona to accept such suburban extensions,* and, in commenting upon the basis upon which such extensions should be made, said:

> But the city will be entitled to a reasonable return on whatever expenditure may ultimately be ordered by the commission, based upon substantial evidence with rational probative force. On this phase of the case it should be noted that a municipal water company may serve residents within the city at cost, but is entitled to a profit on service in territory outside its boundaries. *Ambridge* v. *Public Utility Commission* (1939) 137 Pa Super Ct 50, 31 PUR NS 50, 8 A2d 429. The complainants in this case will not necessarily be obliged to bear the entire cost of extensions of proper mains for service to their properties, but they may be required

*88 PUR NS, p. 371.

to pay the city a reasonably profitable return on the investment involved. It is not unusual for a municipality under such circumstances to require security assuring an adequate return until such time as additional users connecting with the mains make the extension self-supporting at rates, as fixed by the commission, or by agreement by the parties.

What factors must be considered to determine the profit the utility should receive, and the amount of investment, if any, the utility may prudently make in a suburban extension? This is the same question which confronts the private utilities in determining extension policy. A principal difference exists, however: in most states the policies of private utilities are determined by rules approved by, or written by utility commissions. In only a few states are municipal utilities bound by the regulations of utility commissions, even in making suburban extensions. A basic assumption may, however, be accepted—that both municipal and private utilities are in suburban areas on a proprietary basis, and their suburban extension policies should be similar.

In Indiana, the extension policies of both publicly and privately owned water utilities are governed by Public Service Commission "Rules and Standards of Service for the Water Utilities of Indiana—Effective July 15, 1942." Utility managements might ask: Are the terms of the ten-year-old rules of the Indiana Public Service Commission proper today in making suburban extensions? They are not acceptable for two principal reasons, the same reasons that cause a majority of extension rules to be obsolete:

1. An investment by the utility in meters, service pipes, and water main extensions equal to six times the revenue to be received cannot be substantiated unless water rates are much higher—even double or triple the rates which have been based upon prewar operating or construction costs.

2. Basing the extension on the cost of 4-in. pipe is generally unrealistic, inasmuch as 4-in. pipe should not be installed where fire hydrant service or further extension may be necessary in the future. To install a larger pipe without charging it against the people being served would be an obvious subsidizing of the suburban by the city residents.

Utility Investments

The water rates to be charged for suburban service have a definite bearing upon the investment the utility can make in suburban main extensions, or in items such as service lines and meters. An investment is warranted only if revenue to carry the investment in suburban facilities is received in excess of the revenue necessary to carry the investment in facilities bringing the supply up to the beginning of the suburban extension.

That water rates outside a city should be higher than water rates inside a city is generally accepted. As the adequacy of water rates has an intimate relationship to extension policy, a few of the factors concerning suburban rates should be mentioned:

1. City rate schedules are based on cost of serving average customers, some located near the center of the city and some near the city limits; the suburban schedules apply to customers who are all outside the city limits, thus requiring a maximum of facilities.

2. City rate schedules of municipal utilities usually contain no charges for facilities which have been paid for by tax assessment or by the application

of surplus accumulation; new suburban customers have made no such payments and their rates should be high enough to make up the difference.

3. The interest rates to be applied against the investment provided by the city water users or taxpayers should not be the interest rate at which the city can borrow money, but should be the value of money to the individuals who paid for the facilities through water rates or taxes, probably an average of 6 per cent. Depreciation should also be included at 1.5 or 2 per cent. As the operation is a municipal one, no property or income taxes are involved.

An analysis might show, for example, that the debt-free value of a municipal water works system is $150 per minimum customer. If the water rates were carefully designed, and showed that the minimum charge inside the city should be $12.00 per year, then the minimum charge outside the city should be at least $11.25 per year higher (7.5 per cent of $150 for interest and depreciation). The annual minimum bill should properly be even higher than $23.25 because the distance factor to the outside customer has not been added. In addition, the rates should be still higher if the utility is to make a water main extension for such a suburban customer, or is to maintain a water main installed at the cost of the suburban customer.

Cost Factors

The procedure involved in determining the cost of a suburban extension should include the following:

1. The cost of supplying water at the city limits, including customer costs, should be determined. This cost will normally be somewhat higher than the amount resulting from the application of the city rate schedule.

2. If the foreseeable suburban water load will not overtax the plant and transmission mains, a determination of the charge for the use of the facilities which have been financed out of water collections or taxes should be made. If suburban customers will require an enlargement of the plant or mains, however, the unit cost of these items at today's higher prices must be figured. Such enlargement is the normal rather than the exceptional requirement.

3. The percentage of revenue remaining after deductions are made for the cost of supplying water and the annual cost for the system facilities being used, or which must be ultimately installed, should be determined. These facilities are not limited to the water main extension, the service pipe, and the meter, but must include extensions of plant and primary mains which must ultimately be enlarged because of the addition of new customers.

4. The amount of investment the utility can justify in an extension, including service pipe and meter if the utility is to supply them, can be determined by dividing the dollars remaining by the annual charge for interest, depreciation, and maintenance. If 20 per cent of the suburban revenue remains and can be applied toward carrying the costs of extending service to a customer who will pay $30 per year, the utility might invest $75 in facilities for extending service ($\$30.00 \times 0.20 \div 0.08 = \$75.00$; interest—6 per cent; depreciation—1.5 per cent; maintenance—0.5 per cent). This justifiable investment must be divided between the facilities to supply the new customer directly—main, service, and meter—and the indirect facilities—plant, primary mains, and other prop-

erty. If secondary mains, services, and meters equal 50 per cent of all property allocable to general service, then 50 per cent of \$75.00, or \$37.50, could be invested in direct facilities for the new suburban customer. This insignificant investment for a fair-sized customer, just one and one-quarter times his annual revenue, could result because of inadequate water rates.

If 40 per cent of the revenue were available for expanding service, then \$75.00, or two and one-half times the annual revenue, could be used to install water main, service, and meter. Little could be used for a water main extension if a free service line and meter were to be furnished by the utility.

There can be no general formula which will be applicable to suburban extensions of all municipal utilities, just as no general extension rule can be equitably applied to all private utilities. Water rates, cost of service, and investments in plant and transmission mains are variables which are different for each utility operation, and all have an effect upon water main extension policy. It would be just as reasonable to say that every man should wear 9B shoes because more than half could wear that size. Each extension policy should be fitted to the utility operation.

An extension rule which is equitable inside a city is probably not equitable outside that city. Assume that a utility imposing a 75¢ per month surcharge for service outside the city had an extension policy calling for an investment equal to four times the annual revenue of a new customer inside the city. If the extension formula were mathematically applied, the utility could invest $6 \times$ \$9.00, or \$54 more for a customer outside the city than it could invest for a customer inside the city. This formula would be unfair inasmuch as a large portion of the surcharge would normally represent the added cost to supply service, and very little would be available to support an extension.

Pipe Size

The size of pipe to be used in a suburban extension is a common problem. Too often the suburban customers are allowed to install any size or material they wish, with the result that pressures become deficient, fire hydrants cannot be connected, or the pipe quickly fails because of corrosion. The utility should insist upon full control over the installation including size, material, and ownership, even if all costs are paid by a customer or real estate promoter. If fire protection and probable further extensions are considered, it would appear that suburban mains should normally be at least 6 in. in diameter.

A provision for returning most of the cost to the promoter, if and when additional customers attach to an extension, is desirable. This feature might be provided by charging a tapping fee for all services and returning the fees to the promoter, together with the investment, if any, the utility could make for each new customer. One municipal utility charges tapping fees of \$150 for a $\frac{5}{8}$- or $\frac{3}{4}$-in. service, \$200 for a 1-in., \$250 for a $1\frac{1}{2}$-in., \$300 for a 2-in. service, up to \$2,000 for a 6-in. service. These fees are collected by the utility and paid to the promoter during the ten years after a suburban main is installed. Refunds, of course, do not exceed the original payment made by the promoter. This method distributes, in a fairly equitable way, the cost of suburban extensions among the persons being served.

Conclusions

The increasing demand for extensions into suburban territory, together with today's high costs of such extensions, makes it imperative that extension rules be revalued and redesigned. Many rules which were equitable ten or fifteen years ago are inequitable today, particularly in their impact upon the pocketbooks of the old customers. Other rules were never properly designed, and today are seriously affecting the economic existence of both private and municipal utilities. Today many utilities and several utility commissions, however, are reexamining and modernizing extension rules.

The elements of water main extension policy have been discussed at length in water supply literature, and have been summarized by the American Water Works Assn. Committee on Water Main Extension Policy (1). With the aid of the knowledge available on the subject, water works managements can effectively overhaul their extension policies so that new customers, even in suburban territory, can be supplied water service at reasonable cost without placing undue burdens, through ever increasing rates, upon the old customers of the utility.

Reference

1. COMMITTEE REPORT. Water Main Extension Policy. Jour. AWWA, **41**: 729 (Aug. 1949).

Discussion

A. O. Norris

Exec. Vice President, Indianapolis Water Co., Indianapolis, Ind.

The writer has always found it difficult to explain satisfactorily to anyone desiring water service involving a main extension what all the expenses are. When the distribution system is extended to provide for new customers, there are cost factors in addition to the cost of the extension itself which must be weighed against the revenue the new customers produce. These other cost factors are often overlooked or not given full consideration—one of the principal, if not the principal, reasons why so many water works systems are now municipally owned. Frequently when property was originally constructed with private capital, the original owners either lost sight of these other expenses when extending the distribution system or were unable to finance such extensions properly. As a result, the privately owned utility found itself unable to meet the growing needs of the community, and acquisition by the muncipality rather than a substantial increase in rates was the solution. Had the rates been increased, the old customers would have had the burden of carrying the newer consumers.

The writer agrees with the author's reasoning that an old customer residing within the corporate limits has a proprietary interest in the municipally owned water works, and that this interest should be protected so that he will not have to pay higher rates to help support the extension of the municipality's system into areas outside the corporate limits. The old customer can be protected, not by refusing to extend into suburban areas, but by properly financing such extensions through the recognition of all the costs.

Cost Elements

The importance of knowing what elements of cost enter into a main extension for a municipally or privately owned water works system cannot be overstressed. Undoubtedly every water works man knows that, along with the growth of the distribution system, other parts of the plant must also grow. But knowing is not enough. It is the writer's belief that the water works management has frequently failed to sell this fact convincingly to prospective customers requiring extensions and to regulatory bodies which have jurisdiction over this aspect of a utility's operation. It is too bad that pumps, filters, basins, and other supply facilities do not come in small pieces and that they are not required additions to the system as mains are extended. The total and true cost of adding customers by main extensions would then become more apparent. The problem lies in the difficulty of recognizing and giving the proper weight to these other facilities, such as pumps and filters, when working out the financial terms for a main extension. For any growing utility, expenditures for plant will have to be made from time to time. To simplify the recognition and inclusion of these other costs in a main extension, a formula could be applied:

$$X = \frac{(A - Ab)C}{R}$$

in which X equals the amount, in dollars, that can be spent on a main extension for the gross revenue, A, that is estimated to be available from the extension. The letter b is the operating ratio of the utility—the percentage of total gross operating revenues that is spent for all operating expenses except the cost of borrowed money. C is also a percentage figure, and should represent that part of the total plant that is invested in pipelines. R represents the rate of return desired, or the earnings needed, expressed as a per cent of the total investment neeeded annually to cover the cost of borrowed money. This formula can be applied to the author's example.

An extension of main is needed to procure a new customer who will provide an estimated revenue of $30.00 per year. How much can the utility invest for this revenue? The utility spends 60 per cent of its gross operating revenues on operating expenses, and its investment in mains is 40 per cent of its total investment in plant and property. Then:

$$X = \frac{(30 - 30 \times 0.60)(0.40)}{0.06} = 80$$

For a rate of return of 6 per cent, the utility can, therefore, spend $80.00 for a customer who will yield $30.00 in revenue.

Looking at this formula from another angle, the b factor reduces the available income from the new customer to the remainder after operating expenses, and the C factor again reduces the revenue to the amount that is available to support the utility's investment in the main extension. Dividing the reduced available earnings by R produces the amount the utility can afford to spend on the extension. By spending this amount and no more, the utility will preserve enough of the added revenue to finance the cost of other facilities which will be needed immediately, or at some later date, for this added customer. The writer believes that the factors making up this formula are self-explanatory and readily available to all water works managers.

If the utility not only extends its mains but also installs the service line, meter, and other service line appurtenances, the formula is still applicable, In arriving at the value for C, however, the total investment in mains, service lines, meters, and other appurtenances, or whatever is actually installed for a new customer, should be related to the total investment in all facilities. The value for C would be nearer unity than it would be if only the cost of mains were considered, and would permit a greater investment in dollars for a new consumer. This does not necessarily mean that a greater number of feet of main could be installed, however.

Pipe Size

Another problem in main extension policy is the size of pipe. Many times a manager of a water works utility is confronted with the problem of asking one developer to finance the cost of a small-diameter main while at the same time he has to require another developer to finance a large-diameter main. This situation arises when the growth possibilities beyond the developments in question vary materially. The writer feels that the equitable solution for such a predicament would be to require each developer to finance an extension on the basis of the same pipe size, regardless of the size actually required. This same pipe size could be the weighted average pipe size of the distribution system, a figure which could be varied to fit recent changes in main extension policies or recent experiences in the sizes of mains installed. The main objective is to treat all developers on the same basis and to have the total of financing for a year truly representative of the cost of the various size mains installed.

Having arrived at the weighted average pipe size, the next step is the pricing of this average size. It would be simple if a distribution system had a weighted average pipe size of, for example, 6 or 8 in., as then the actual current cost of installation could be used. If, however, the average pipe size turns out to be 7 in., the current installed cost of various size mains could be plotted against the square of their diameters. The writer has found that such a curve is almost a straight line and, therefore, it is a simple matter to arrive at a price of a weighted average pipe size. It would only be necessary to locate the point 49 (7 squared) on the curve and find the corresponding unit price. This hypothetical unit price for calculating the costs of all main extensions would place the financing of all main extensions on the same basis in so far as the cost of the pipe is concerned. The actual amount of pipe the utility could afford to install under such a plan would, of course, depend on the additional revenue available, and would be determined by first ascertaining the dollar amount it could invest for the added revenue, and then dividing by the hypothetical unit price figure.

Use of Magnesium Anodes in Cathodic Protection of Tanks and Pipelines

By L. E. Magoffin

A paper presented on Oct. 25, 1951, at the California Section Meeting, San Francisco, Calif., by L. E. Magoffin, Asst. Engr., California Water & Telephone Co., National City, Calif.

THE corrosion of metallic structures in the water works industry has been a serious and costly problem for many years. The annual loss in pipelines alone has been estimated at $200,000,000. Of the many methods of correction applied to this problem, one of the most successful has been cathodic protection.

Several methods of applying cathodic protection have been devised, among them being use of the galvanic anode. Galvanic anodes were first introduced by Sir Humphry Davy in 1823, when he suggested the use of zinc anodes for the protection of ship bottoms. Since that time, many metals have been used, with magnesium being among the most successful for mitigating steel corrosion.

It has long been known that when two dissimilar metals are placed in a common electrolyte and connected electrically, a current will flow between them. Such a system is known as a galvanic cell. Every metal has a definite tendency to dissolve, or go into solution, when placed in a given electrolyte. This tendency is technically referred to as a solution potential. When two metals having different solution potentials are connected in a common electrolyte, the metal with the greater potential will dissolve preferentially and, in so doing, will inhibit the solution of the metal having the lesser potential. Electrochemically, the ions liberated by the dissolving particles of the metal having the higher solution potential (the anode) form an electric current which flows through the electrolyte to the metal having the lower solution potential (the cathode). This current acts to inhibit the solution of the metal to which it is flowing and thus cathodically protects it. The greater the difference in solution potential between the two metals, the greater will be the current flow between them. Magnesium has the greatest solution potential of all commercially available metals, thus making it ideal for cathodic protection use (1).

Application of Magnesium Anodes

In 1946 a cathodic survey was made on three large transmission mains of the California Water & Telephone Co. From the survey, it was learned that much could be accomplished toward mitigating corrosion on these mains by cathodic protection.

Several rectifier type cathodic stations were installed on these mains with excellent results. It was soon realized, however, that the entire length of the mains could not be protected with rectifier type stations because of the highly developed residential areas they traversed. Lack of locations for anode beds and underground pipelines of other

utilities made it necessary to look for some other means of cathodic protection. Magnesium anodes seemed to be the logical answer.

The first magnesium anodes were installed on 735 ft of 30-in., No. 10 gage riveted-steel pipe laid in 1922 in "E" Street in Chula Vista, Calif. The pipe had one layer of asphalt-felt wrap. Upon examination, the wrap was found to be in very poor condition. For purposes of design, the pipe was estimated to be 35 per cent bare. From the survey made in 1946, the current requirement was estimated at 1.16 ma per sq ft. Under given conditions, the estimated output per anode was 180 ma. It may be computed from the equation:

$$\frac{\text{pipe surface area} \times \text{current density}}{\text{current flow per anode}} = \text{number of anodes required}$$

that twelve anodes would be required to bring the pipe-to-soil potential to -0.85 v measured to a $Cu \cdot CuSO_4$ electrode.

Twelve 8-ft holes were augered at 60-ft intervals at right angles to the pipeline and 10 ft from it. A 32-lb magnesium anode packaged with special backfill material was placed in each hole and was connected to the pipeline with a No. 10, insulated, copper wire preattached to the anode. Split-bolt connectors were placed in the lead wires for testing purposes.

A pipe-to-soil potential and current-output survey was made after all the anodes were installed. The average output per anode for the entire installation was 180 ma. The average pipe-to-soil potential was -0.468 v before protection and -0.726 v after the installation was completed. Potential measurements were made with a potentiometer, and current measurements were made with a milliameter.

Field tests have indicated that, under normal conditions, approximately 500 amp-hr will be realized per pound of magnesium (2). On this basis, a 32-lb anode such as was used in this installation, draining at the rate of 180 ma, will have a ten-year life.

Another Installation

In the early part of 1951, a visual inspection of a 10-in. bare steel main in Cleveland Avenue and 15th Street in National City, Calif., was made. The total length of pipeline was 1,064 ft. This main was laid in 1943 by a shipbuilding company, with no particular attention being given to protection of the main from corrosion. The main was later acquired by the California Water & Telephone Co. Severe corrosion was noted during the inspection which was made at various easily accessible points along the main.

Soil conditions varied from adobe clay to dredged sand with salt water sloughs. Soil resistance ranged from 1,000 ohms per cu cm to 250 ohms per cu cm. Pipe-to-soil potentials ranged from -0.385 v to -0.450 v.

The mechanics of installation of a cathodic protection station here were complicated by surface structures such as railroad tracks, high industrial fences, and other obstructions, as well as by the underground pipelines of other utilities. Again, magnesium anodes seemed to be the logical answer to the problem.

As in the "E" Street installation, an estimated current density of 1.16 ma per sq ft of pipe was used. Under given conditions, the estimated output per anode was 280 ma. From the above equation, it may be computed that twelve anodes would be required to bring the pipe-to-soil potential to -0.85 v.

Twelve 50-lb anodes were installed in the same manner as those on the "E" Street installation. The 50-lb anodes were used to obtain as long a life as possible in the low-resistance soil. On the basis of 500 amp-hr per pound of magnesium, the anodes, draining at the rate of 280 ma, will have a life of approximately ten years.

A pipe-to-soil potential and current-output survey was made in July 1951. The average output of the anodes located in sloughs and dredgings was 395 ma, 115 ma more than the anticipated output, whereas the output of the anodes located in clay and adobe soil was 230 ma, 50 ma less than expected. This difference was apparently caused by changing soil resistivity. The average pipe-to-soil potential was − 0.83 v.

Protection of Tanks

Magnesium anodes have been used successfully for the mitigation of corrosion on the inside and outside of steel tanks (3, 4). The basic design of cathodic protection on steel tanks, either inside or outside, is similar to that for steel pipelines. Several factors which are not encountered in the design of stations on subsurface structures, however, must be faced. Some of them are:

1. *Precise location of anodes on the inside of tanks.* Anodes must be properly spaced inside a tank to obtain necessary current distribution on all exposed surfaces.
2. *Weight.* Anode weight loads, when suspended from the roof of a tank, must be carefully controlled.
3. *Varying resistances of waters in water storage tanks.* In many water distribution systems, several sources, the waters of which have a considerable difference in resistance, are used. When designing a cathodic protection system, the most adverse possible condition must be used as the basis of computations.
4. *Static water level inside tanks.* If the water level does not remain in contact with the anodes long enough for polarization to take place on the exposed surfaces of the tank, little can be accomplished with cathodic protection.

In September 1950, a bolted-joint, steel water storage standpipe was installed in National City, Calif. The tank was 16 ft high and 21½ ft in diameter. The inside of the tank was painted with one coat of red lead paint. Owing to the corrosiveness of the water, cathodic protection was planned from the beginning.

Rectifier type cathodic protection with platinum and carbon anodes had been used in water tanks in the area with good results. Magnesium anodes had been considered but, because of lack of information and experience, had not been used before this tank was installed. The tank was small and the initial investment, therefore, would be low. If magnesium anodes proved unsatisfactory, little would be lost, and good experience would be gained. On this basis, magnesium anodes were installed.

From previous experience, a current density of 1 ma per sq ft was used to estimate the current requirement. The water resistance was 961 ohms per cu cm. Table 1 gives a chemical analysis of the water. Six sections of 1,315-in. diameter iron-core, magnesium rod, 13 ft long, were used for anodes. The spacing of the anode rods is shown in Fig. 1. Under given conditions, 225 ma output per anode could be expected. Each anode rod had a weight of 16.5 lb. On the basis of 500 amp-hr

per pound of magnesium, the anodes should have a life of approximately four years.

To date, no attempt has been made to measure the output of the anodes. On July 6, 1951, however, the tank was drained, and a visual inspection of the interior was made. The red lead paint was found to be in very poor condition. In many places it had completely sloughed off, leaving the bare metal exposed, but no evidence of corrosion was found, and the steel surface was in perfect condition. In some areas, a thin, white film had deposited on the surface of the steel. The magnesium rods were in good condition, with a small amount of corrosion evident.

All the installations mentioned in this discussion were made by regular pipeline crews with no special training. Table 2 gives a cost comparison

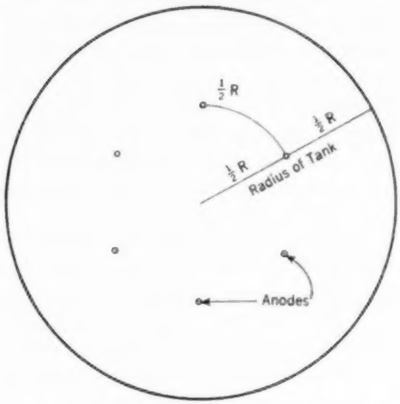

Fig. 1. Spacing of Magnesium Anodes in Tank at National City

The anodes were made of 1.315-in. diameter, iron-core, magnesium rods. Each rod weighed 16.5 lb.

TABLE 1

*Chemical Analysis of Water in Tank at National City, Calif.**

	Concentration
Gravimetric Analysis	*ppm*
Silica	20
Iron	0.3
Manganese	—
Aluminum	—
Fluoride	0.3
Boron	—
Carbon dioxide	8
Calcium	38
Magnesium	23
Sodium & Potassium	167
Bicarbonate	297
Carbonate	0
Chloride	183
Sulfate	52
Nitrate	0
Total solids	630
Total hardness as $CaCO_3$	189
Carbonate hardness as $CaCO_3$	189
Noncarbonate hardness as $CaCO_3$ (Negative)	54
Reacting Values	*milliequivalents*
Calcium	1.897
Magnesium	1.892
Sodium	7.337
Bicarbonate	4.868
Carbonate	0
Chloride	5.175
Sulfate	1.083
Nitrate	0
Concentration value—*milliequivalents*	22.252
Properties	
Electrical conductance, ($K \times 10^5$ at 25 C)	104
Hydrogen ion concentration, pH	7.8
Langelier index at 20 C	0.0
Sodium—*per cent*	65.9

* Water taken from National City well No. 1 on August 6, 1951.

TABLE 2

Comparison Between Cathodic Protection of Tanks With Rectifiers and With Magnesium Anodes

Item	Magnesium Station	Rectifier Station
Output, *amp*	1.35	5.00
Estimated life, *yr*	4	40
Total cost	$85.00	$680.00
Annual capital charges:		
Interest at 6%	$ 5.00	$ 41.00
Taxes at 1½%	1.00	10.00
Depreciation	21.00	17.00
Total	27.00	68.00
Maint. and operation:		
Inspection	10.00	21.00
Maintenance	—	15.00
Power	—	28.00
Total	10.00	64.00
Total annual cost	$37.00	$132.00
Area protected, *sq ft*	1,302	5,497
Cost per 100 sq ft per yr	$2.84	$ 2.40

TABLE 3

Comparison Between Cathodic Protection of Pipelines With Rectifiers and With Magnesium Anodes

Item	Magnesium Station	Rectifier Station
Output, *amp*	2.16	38.00
Estimated life, *yr*	10	40
Total cost	$403.00	$1,924.00
Annual capital charges:		
Interest at 6%	$ 24.00	$115.00
Taxes at 1½%	6.00	29.00
Depreciation	40.00	48.00
Total	70.00	192.00
Maint. and operation:		
Inspection	12.00	21.00
Maintenance	—	130.00
Power	—	127.00
Total	12.00	278.00
Total annual cost	$ 82.00	$470.00
Area protected, *sq ft*	5,770	74,606
Cost per 100 sq ft per yr	$ 1.42	$ 0.63

between cathodic protection of tanks with rectifier stations using carbon and platinum anodes and cathodic protection of tanks using magnesium anodes. Table 3 offers a cost comparison between cathodic protection of pipelines with rectifier stations using steel railroad rail anodes and cathodic protection of pipelines using magnesium anodes. These tables were developed on the basis of experience in the area covered by this discussion.

Conclusions

Although the overall cost of magnesium for cathodic protection has proved slightly higher than that of rectifier type cathodic protection, as shown in Tables 2 and 3, its ease of handling and adaptation have more than offset the additional cost. In areas where power is not available or rights-of-way are difficult to obtain, magnesium provides a good means of cathodic protection. Although magnesium is limited to low-resistance electrolytes, many of the soils and waters in California are of low enough resistance to make its use possible.

Cathodic protection of tanks with magnesium, although considerably higher in cost per square foot, often appears justified because of its light weight and ease of handling. Where

roof loading of tanks or suspension is a problem, magnesium is greatly appreciated. A two-man crew can usually make the complete installation.

Although −0.85 v is the commonly accepted pipe-to-soil potential, it is often not economically feasible to maintain a potential this high. Many pipelines such as those mentioned in this article were old and seriously corroded before protection was installed. Although the computations were based on a pipe-to-soil potential of − 0.85 v, the actual potential was less than anticipated. This factor has been attributed to the unusually dry period which has prevailed in southern California since these installations were made.

If it is found, after installation of magnesium anodes, that the desired potential has not been reached, it is a fairly simple process to compute the number of additional anodes required to bring the line to the desired potential. If, however, the potential is too high, resistors can be placed in the anode lead wire of the necessary value to bring the line to the desired potential.

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Corrosion and Cathodic Protection of Pipelines

By W. R. Schneider

A paper presented on Oct. 25, 1951, at the California Section Meeting, San Francisco, Calif., by W. R. Schneider, Asst. Engr., Bureau of Tests and Inspection, Pacific Gas & Electric Co., Emeryville, Calif.

THIS discussion of corrosion will be confined to the installations of continuous steel pipelines under water or in soil at normal soil temperature.

The corrosion of underground metallic structures at normal temperature is explained satisfactorily by the electrolytic theory. An electric current leaving a metallic surface to enter the soil or water electrolyte with which the surface is in contact will cause a loss of metal at the point of departure —the "anodic area." The surface on which the current reenters the metal, known as the "cathodic area," will show little or no sign of corrosion. These anodic and cathodic areas may be separated by only a small fraction of an inch, causing the individual pits characteristic of local corrosion, or they may be many miles apart, causing "long-line" current to flow in the pipe.

Local currents are caused by such differences, in the metal surfaces of two adjoining areas, as those in crystal size, strain, weld, contact with dissimilar metals, carbon or coke; and in the soils or water with which the surfaces come in contact, as those in salts, concentrations of salt, types of soil, presence of minerals, and concentrations of air and oxygen penetrating the soil. Conditions producing local corrosion are shown in Fig. 1.

Long-line currents are found in pipelines crossing two or more types of soil, which, in addition to having natural differences of potential, may contain different amounts of moisture, dissolved salts, minerals, oxygen, or other materials. This condition is found in irrigated fields adjoining grazing lands, pipes laid partly under open fields and partly under paved areas or from an open field to a submarine crossing, as shown in Fig. 2.

Only comparatively small long-line currents are carried by cast-iron pipes with bell-and-spigot type couplings, sealed with oakum, lead, cement, or sulfur, because of the high electrical resistance of these jointing materials. Thus, the failure of cast-iron pipe is generally considered "local corrosion."

The corrosive areas along a pipeline may be located by various methods, including: soil resistance tests; pipe-to-soil potential tests; pipeline current tests; soil-surface potential tests; and earth current tests.

Soil Resistance Test

Soil resistances are measured by obtaining soil samples at pipe depth at various points along the line and measuring the specific resistance with a standard soil box or in a Putnam (1) block. The four-electrode method, requiring a voltmeter and ammeter, a

megger (megohmmeter), or a McCollum earth current meter,* is used to determine the average soil resistances at various depths by means of surface measurements.

Soils are grouped in four classes of corrosivity according to their specific resistances, as shown in Table 1. Low resistance soils generally indicate the presence of moisture and soluble salts. If the pipe is bare or if its coating is poor or defective, corrosion will result if the pipe potential is higher than that of the contiguous soil.

When soil resistance tests are made, the type of soil must be noted. A compact colloidal formation—such as clay, adobe, or gumbo—may have a comparatively high specific resistance, but, because a lesser amount of air and oxygen can penetrate to the surface of the pipe, it produces a more corrosive condition than many other layers of lower resistance soils.

TABLE 1

Classification of Soils by Corrosivity

Class	Resistance *ohms per cu cm*	Corrosivity
1	More than 5,000	Little or none
2	1,000 to 3,000	Mild
3	500 to 1,000	Moderate
4	Less than 500	Very

Resistance, actually, is only one of the soil characteristics that influence the underground corrosion of metals. The amount of current discharged from a pipe is also influenced by the resistance of the pipe wrapping and the potential of the pipe.

Pipe-to-Soil Potential Tests

The pipe-to-soil potentials are measured with a high resistance voltmeter or a potentiometer connected between the pipe and a copper sulfate or other electrode placed close to the pipe. The potential gradient between a copper sulfate electrode and a steel pipe that is neither collecting or discharging current into the soil has been found to vary from 0.49 v to 0.76 v, a fair average value being 0.60 v. This potential is ordinarily accepted as a basis for determining the polarity of a pipe relative to the surrounding soil (2).

The pipe-to-soil potentials indicated by the voltmeter are a combination of the iron-to-copper sulfate galvanic couple and the unknown contact potential between the copper sulfate and the particular type of soil at the test location, the drop of potential in the soil between the pipe and electrode caused by a current flowing to the pipe or across country, and the polarization potential of the pipe. When the potential measured to a copper sulfate electrode is less than 0.60 v it is assumed that the pipe is discharging a current into this soil. If the potential is greater than 0.60 v, the assumption is that the pipe is collecting current

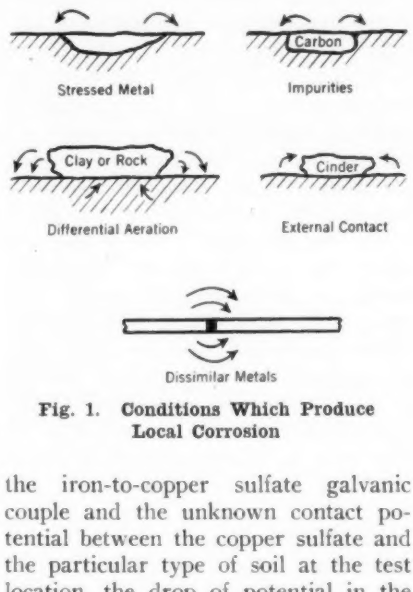

Fig. 1. Conditions Which Produce Local Corrosion

* The McCollum meter is described under "Earth Current Surveys," p. 417.

from the soil. If a potential of 0.85 v or more is found, it is assumed that corrosion has been eliminated entirely by the current received through the electrolyte.

The existence of a pipe-to-soil potential does not necessarily indicate a corrosive condition. The resistance between the pipe and soil may be very high because the pipe insulation and the dryness of the soil prevent the flow of a current, the meter merely indicating the potential of the local galvanic couple produced by the temporary presence of the copper sulfate electrode near the steel pipe.

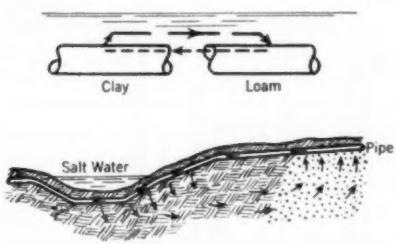

Fig. 2. Types of Long-Line Currents

Measurement of Pipeline Currents

The direction and magnitude of the current flowing in a pipeline can be found, for the purpose of determining the corrosive areas, by measuring the drop of potential over a given length of pipe—usually 100 to 500 ft—with a voltmeter or potentiometer. The current is then calculated from the known resistance or weight of the pipe (2).

The most satisfactory method of connecting the copper potential leads to a pipe is by riveting the copper wire into a $\frac{1}{16}$-in. hacksaw groove under a lip raised with a cold chisel, or by welding the lead to the main. The connection should always be covered with several shovelfuls of soil to avoid the creation of thermal potentials on the meter circuit. The use of contact points and probe rods of various kinds, in addition to the thermal potentials, introduces galvanic potentials created by an unnoticed film of moisture, soil, or other foreign matter between the apparently clean contact points and the steel pipe.

When the weight of the pipe is unknown, due to the loss of original records, later unrecorded repairs and renewals, or use of various available sizes due to material shortages, tests are made to determine the resistance of the pipe by the Hering method (3) shown in Fig. 3. The difference in the fall of potential over a known length of pipe before and after a measured current is sent through the line from an outside source (such as a battery), when divided by the superimposed battery current, will give the resistance of the pipe between the voltmeter connections on the pipe.

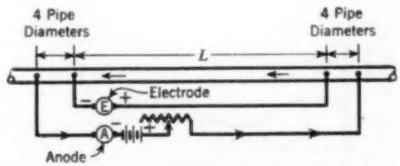

Fig. 3. Hering Divided Circuit for Determining Pipe Resistance

The pipeline currents are plotted as shown in Fig. 4. Any decrease of current in the direction of its flow indicates a discharge to the surrounding soil, proper allowances being made for all service connections, branch lines, and accidental contacts in the tested area. In Fig. 4, the portions of the line between *B* and *C*, and between *E* and *D* are losing current and require further investigation for the application of protective measures. These anodic portions are usually in such

areas as irrigated lands, rice fields, swamps, under streams, former tide-lands, clay formations, and under pavements. The portion of the line collecting current from the soil, which is generally the current discharged in the corroding areas, may be on higher ground, grazing land, orchards, grain fields, and other comparatively dry areas.

Potential Gradient Survey on Soil Surface

A method of locating anodic or corroding areas on a pipeline by measuring the potential drop on the surface

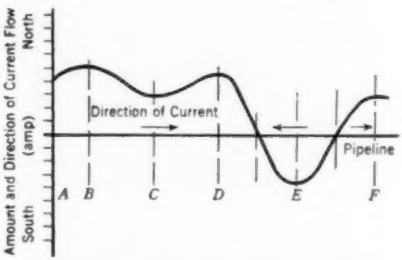

Fig. 4. Plot of Pipeline Currents

Losses of current are shown between Points B *and* C *and Points* E *and* D.

of the soil along the line has been described by O. C. Mudd (4). The potential drops measured are caused by the actual corrosive current flowing in the soil and are not merely indications of "possible" corrosive areas indicated by the pipe-to-soil potential tests or by soil resistance tests.

In the soil surface test, the potential drops are measured between a series of points on the pipe, approximately 1,000 ft apart, and copper sulfate electrodes placed on the surface of the soil directly above each point. The potential gradients caused by the current flowing in the soil from the anodic to the cathodic areas are then measured on the surface of the soil above the main between two copper sulfate electrodes spaced at intervals of approximately 100 ft. These surface gradient increments are added to the initial pipe-to-soil potential found at each original station. The potential indicated at each surface test location is the pipe-to-soil potential caused by the current flowing in the soil between the two portions of the pipe surfaces. It is not necessary to make contact with the pipe at the intermedi-

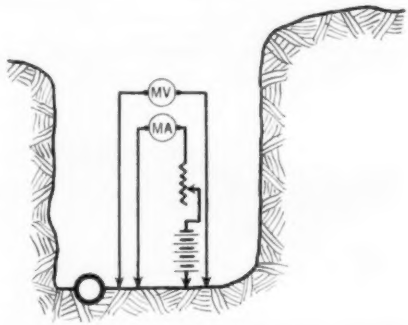

Fig. 5. Diagram of McCollum Earth Current Meter

Application of instrument to measure the soil current flowing to and from pipe is shown.

ate points. When these values are plotted as a potential profile, they will show a decided peak at the anodic portions of the line. At such places the spacing of the electrodes can be reduced from 100 ft to 4 or 5 ft or less, making it possible to locate the center of a discharging area within a few feet.

The advantages claimed for this type of test are that less time and labor are required to cover a given length of line and that the accuracy is greater than that attained by other methods. Certain precautions are re-

quired to avoid errors previously mentioned as likely to be incurred when copper sulfate electrodes are used in soils of varying types.

Earth Current Surveys

Surveys of the current flowing in the soil, to or from a pipe, are made with the McCollum earth current meter developed by the National Bureau of Standards, Washington, D.C.* This instrument consists of a high resistance millivoltmeter with two copper sulfate electrodes, a milliammeter, two copper electrodes, a battery, and a commutator for reversing the direct current. Schematically, the application of this instrument to measure the soil current flowing to or from a pipe is shown in Fig. 5. The millivoltmeter measures the current flowing in the soil by measuring the potential drop between two copper sulfate electrodes, 4½ or 9 in. apart. This reading is converted to milliamperes per square foot of pipe surface by comparing it with the reading obtained when a known current from a battery is sent through the soil.

When the pipe-to-soil currents are measured along a bare pipeline, the readings will give the correct values and locations of the anodic and cathodic areas and the severity of the corrosion, from which information the probable time of the pipe failure can be calculated. Earth current readings taken by placing the McCollum electrode against the wrapping on a wrapped line will be low in value, and a number of readings will be required in each excavation because of the nonuniformity of the resistance of the usual coatings. Most of the wrappings in moist soil or in water for a number of years will absorb enough moisture to produce many comparatively low resistance areas, but tests on or through a wrapping will not indicate the severity of the corrosion that may exist at some other place where the coating has been injured or stripped off during installation.

* Manufactured by the Rawson Electrical Instrument Co., Cambridge, Mass.

To determine the maximum current flow from a pipe at a bare spot in a given area, under conditions similar to those existing at the time of the test, a portion of the wrapping may be removed, the pipe thoroughly cleaned, soil from the excavation at

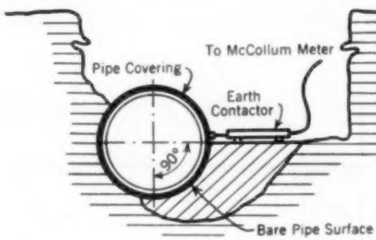

Fig. 6. Earth Current Measurement at Bare Spot on Pipe

To simulate actual conditions, wrapping has been removed, pipe thoroughly cleaned, and soil from excavation at pipe depth packed against bared metal.

pipe depth packed against the bared metal, and the earth current electrode placed as shown in Fig. 6. The results of the earth current meter test made on the pipeline shown in Fig. 4 are plotted in Fig. 7.

Protective Methods

The flow of corrosive current can be reduced or prevented by interrupting the continuity of the electric circuit, either in the pipe or between the pipe and soil. Long-line currents can be reduced by the introduction of insulating fittings in the line between

the anodic and cathodic areas when this boundary is at a fixed location such as a river bank. When placed at locations other than at a definite and fixed boundary between the anodic and cathodic areas, however, insulating couplings usually cause severe corrosion immediately adjacent to the insulating flanges unless a drainage resistance bond is installed to equalize the potential on both sides of the insulated fitting.

The circuit can be broken between the pipe and soil by insulating the main with various materials available for this purpose. This will reduce the local corrosion and, to a large extent,

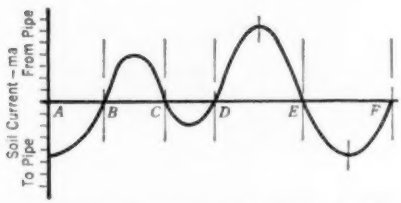

Fig. 7. Results of Earth Current Meter Test on Same Pipeline as Used for Test in Fig. 4

the effects of the long-line currents. Coatings used for this purpose are made of asphalt and coal-tar and of various types of fabrics impregnated with these materials. For very corrosive environments heavier coatings are used, such as the various types of "mastic." Concrete coatings are used on water lines on many systems for both internal and external protection of steel pipes.

The desirability of applying electric protection requires a comparative study of the costs of the various other methods of protecting the line, such as rewrapping or removing the line to a new location. The cost of maintaining an old line may be excessive because of the location and distance over which new material, men, and equipment must be transported; construction difficulties if the line is under pavement, buildings, swamp lands, or is part of a submarine crossing; or intangibles, such as interference with traffic and business, and the possible loss of the public good will, if the work must be carried out in a business district. A diagram showing schematically the method of applying electrical protection to a pipeline is shown in Fig. 8.

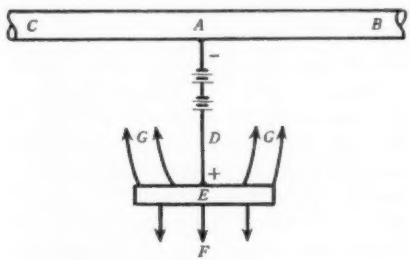

Fig. 8. Method of Applying Cathodic Protection

Electric Protection

The first requirement for the installation of any type of electric protection is a determination of the curliamperes per square foot of bared pipe surface in the soils or waters of various resistances to neutralize the detrimental effects of the discharged long-line currents as well as the local corrosion of the soil. The required current density is given by the empirical equation:

$$i = \frac{1{,}000}{r}$$

in which i is the current required for complete protection of the pipe, in mil-

liamperes per square foot of bared pipe surface; and r is the specific resistance of soil, in ohms per cubic centimeter, as determined from a sample taken at pipe depth. This equation, which includes a factor of safety of 3, is based on a series of laboratory tests on salt water at 18 C, ranging in resistance from sea water of 22 ohms to 5,000 ohms per cu cm. These currents are measured on a McCollum earth current meter as described in the foregoing pages. A steel pipe is generally assumed to receive full electrical protection when the potential to a copper sulfate electrode is 0.85 v or greater.

The test electrode should be placed in contact with a soil stratum at the level of the bottom of the pipe. This is the undisturbed formation on which the pipe was installed and, under a large main, contains more moisture than the upper layers of soil. The pipe-to-soil potential indications may give erroneous information because of the various unknown potentials previously mentioned. As noted in the bulletin of the National Bureau of Standards (5) "such measurements often given erroneous indications as to whether a pipe is discharging or receiving current from the earth." The same statement applies to potential readings taken from pipe to soil when electric protection is applied.

The second step in the installation of electric protection is to determine by test the amount of current, I, that must be impressed on the pipe through the soil from a given d-c source to obtain the minimum required current per square foot of exposed pipe surface at the limits of the corrosive area, as determined by the soil resistance tests. Current from the positive terminal of a battery or portable generator is passed through an ammeter to a temporary ground rod, or bed of ground rods, located 100 ft or more from the pipe, from which it travels through the soil to the pipe, and returns to its source through a cable connecting the pipe to the negative terminal of the battery. The current is increased until the earth current meter shows that full protection is being received over the required length of pipe.

The next step is the selection and design of a suitable anode—those most commonly used being steel, carbon, and the galvanic electrodes, such as zinc, aluminum, and magnesium. Anodes, of either horizontal or vertical steel pipes, are designed to have a minimum weight of 25 lb per amp-yr. This allows a 25 per cent margin for the scrap which will remain in the ground when the anode has disintegrated and is no longer serviceable. The anodes are designed to carry the full output of the generator or rectifier supplying the power for the protection unit.

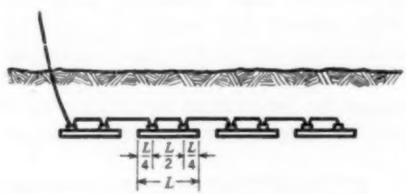

Fig. 9. Horizontal Steel Electrode

Cable connections to each length of pipe at points 8 or 10 ft apart are shown.

The minimum weight of steel required for a 20-amp unit with an anode life of 15 years, will be:

20 amp × 15 yr × 25 lb = 7,500 lb, which is the weight of 150 ft of 12-in. pipe. This weight of steel is the minimum to be used in any soil for an in-

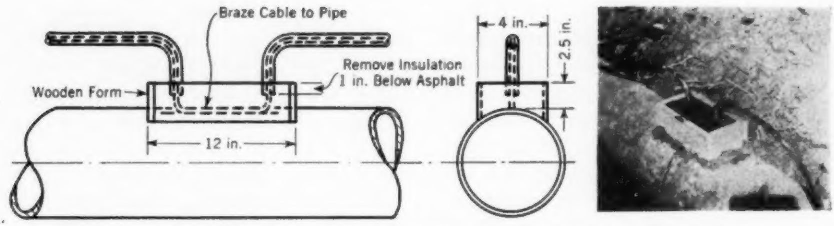

Fig. 10. Insulation of Cable Connection of Horizontal Steel Electrode

Electrode is new or reclaimed steel pipe; cable is 250–MCM underground type.

stallation of this size. Less will be called for by the charts for a given anode resistance in soils having a resistance of only a few hundred ohms per cubic centimeter, and more will be required in high resistance soils, but never should less metal be installed than called for by the above equation.

The resistance of an anode to soil cannot be determined by either area of pipe or weight alone, as the anode resistance is affected by the distribution of the surface in contact with the soil. A smaller pipe, laid in a straight line, will have a lower resistance than a larger diameter but shorter pipe having the same external area. A high surface to weight ratio is required of a conductor to obtain a satisfactory low-resistance ground, and this requirement explains why steel rods and rails are not as economical as pipe for this

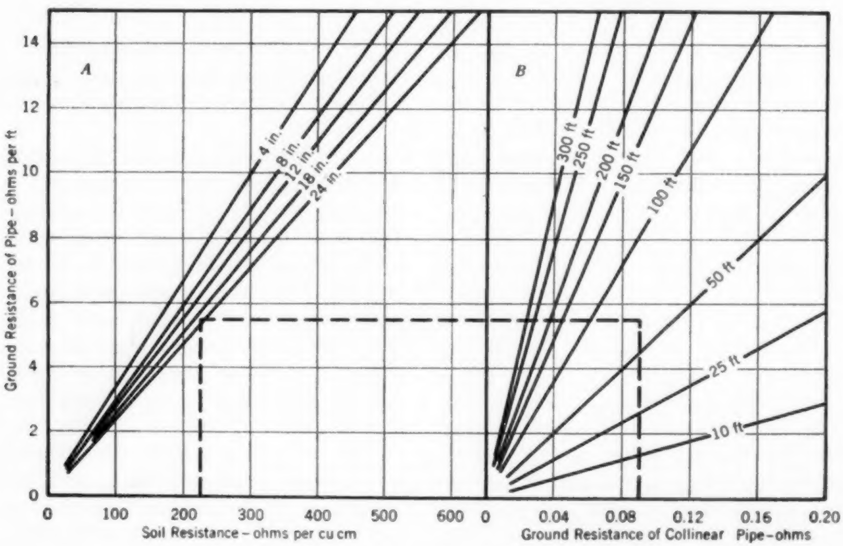

Fig. 11. Resistance of Horizontal Ground Electrodes

Part A *shows variations with different outside diameters of steel-pipe electrodes; Part* B, *variations with their length.*

purpose. In high-resistance soil an appreciable reduction in the weight of steel may be attained by using a lighter weight of pipe.

The selection between vertical and horizontal rods or pipes for an anode must be based on the available size and location of the right-of-way and the soil formations which are encountered. The difficulties of installation are frequently deciding factors.

Horizontal Anodes

Horizontal anodes consist of pipe laid in a straight line at a depth that will insure a moist surrounding throughout the year. Their advantages are:

1. The entire length of the anode, being in moist soil throughout the year, discharges a current uniformly over its entire surface.
2. The anode can be placed in a machine-dug trench.
3. Horizontal anodes can be placed in comparatively shallow soil when rock formations are encountered at greater depths.
4. The anodes and cable connections can be examined with only a slight amount of excavating.
5. The anode is placed underground at a sufficient depth to permit unobstructed use of the surface for farming and other land use.

The horizontal electrode can be placed in any position relative to the pipeline, provided that no part of the anode is within 100 ft of it. A greater spacing increases the length of line receiving protection. The cable is connected to each length of pipe at points 8 or 10 ft apart, as shown in Fig. 9, and the connection is insulated as shown in Fig. 10. Vinyl or Neoprene protected cable is recommended because ordinary rubber insulation deteriorates rapidly in some soils.

When, in order to avoid cathodic interference with other structures, the potential drop in the anode bed is limited to a value such as 1 v (at the full load output of the direct current source), the resistance of the anode bed can be calculated by Ohm's law and the amount of horizontal steel pipe required found by the use of Fig. 11. If, for example, a horizontal steel anode is to be installed in soil having a specific resistance of 1,000 ohms per cu cm, the calculation will be based on:

Current discharged, *amp*	25
Maximum permissible anode potential drop, *v*	1.5
Life of anode, *yr*	20
Minimum weight of anode (25 lb per amp-yr × 20 amp × 20 yr), *lb*	12,300
Equivalent weight of 12-in. pipe, *ft*	215
Anode-to-soil resistance (25 amp at 1.5 v), ohms	0.06

Determination of the weight of anode required may be made from Fig. 13. As the resistance of 0.06 ohm is $\frac{1}{10}$ of the scale shown in the chart, the length is found by multiplying by 10. By this method, it is found that 450 ft of 12-in. pipe, or 25,660 lb, will be required. This amount, although more than the minimum required weight, is the minimum length that must be laid in a straight line to obtain the required low resistance to ground. Some saving may be obtained if a lighter weight of 12-in. pipe is obtainable.

Vertical Electrodes

Vertical electrodes are used in locations having a moist soil formation of sufficient depth and area to permit the required number of anodes spaced to

give a low anode bed resistance, to be installed. Resistance of a vertical anode bed is found by using Fig. 12.

Vertical anodes have the following advantages in certain locations:

1. They are readily installed in soils that are reasonably free of stones or boulders.

3. They can be installed in restricted quarters, in parkways, and between buildings that would usually be endangered by deep trench excavations.

The average resistance of the soil in which a vertical anode is to be installed is found by obtaining samples

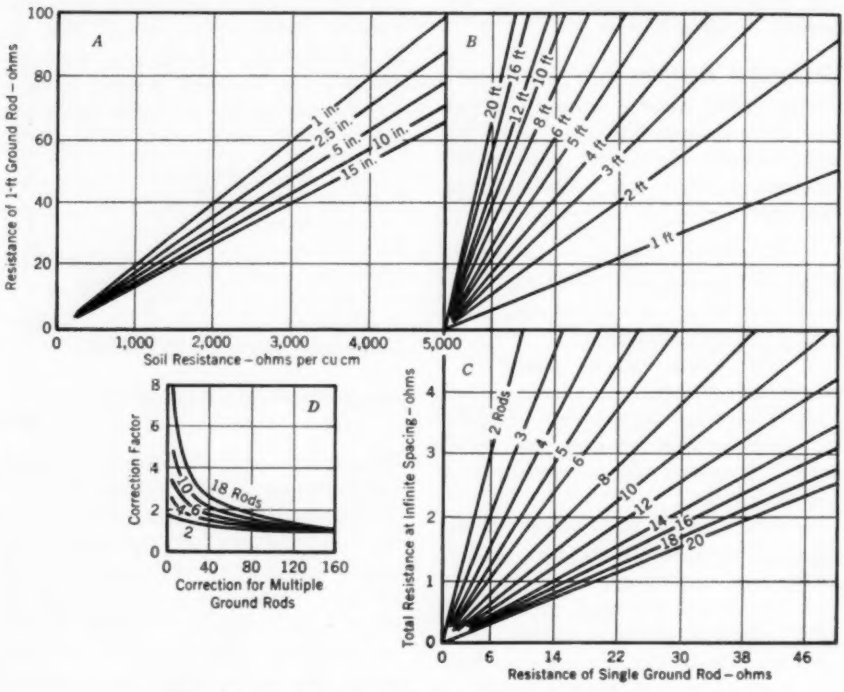

Fig. 12. Resistance of Vertical Ground Electrodes

Part A *shows variations with different diameter rods; Part* B, *with single rods of different lengths; and Part* C, *with number of rods. Part* D *is spacing correction curve. When spacing of vertical rods is less than 250 diameters, total resistance must be multiplied by spacing correction factor.*

2. They can be installed in marsh or flooded areas, where digging or trenching machinery could not be used and where excavations would require the use of pumps and cribbing to avoid cave-ins.

at various depths and calculating the resistance by the equation:

$$R_{sp} = \frac{n}{\dfrac{1}{R_1} + \dfrac{1}{R_2} + \dfrac{1}{R_3} \cdots \dfrac{1}{R_n}}$$

in which R_{sp} is the average specific resistance of the soil; n is the number of samples taken between the surface and bottom of the electrode; and R_1, R_2, and R_3 are specific resistances, in ohms per cubic centimeter, of the samples taken at the depths 1, 2, 3.

Vertical ground rods should be spaced about 250 diameters or more apart to reduce the "interference" or "blocking" between them. If spaced more closely, the total resistance of the bed can be estimated by means of the spacing correction chart (Fig. 12).

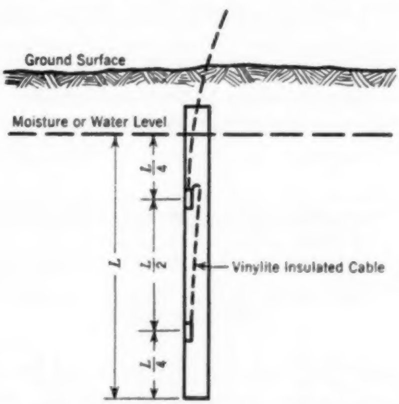

Fig. 13. Vertical Steel Electrode in Place

When the allowable drop of potential at the anode has been determined, the anode resistance can be calculated and the amount of steel pipe required found by use of Fig. 12.

A vertical steel anode will corrode most rapidly at the ground water level, thereby disconnecting the lower portion of the anode before it has served its useful life. This difficulty is avoided by making internal connections to the pipe, about 8 ft apart as shown in Fig. 13 and 14.

Details of an internal connection in a vertical pipe anode as shown in Fig. 14 are made by brazing the cable to a steel disk which, in turn, is brazed or welded to a short piece of 2-in. pipe, forming a cup. This pipe is then welded to the inner surface of the metal "window" cut from the pipe anode. The cup is filled with asphalt and the entire assembly replaced in the anode pipe, as shown in Fig. 15.

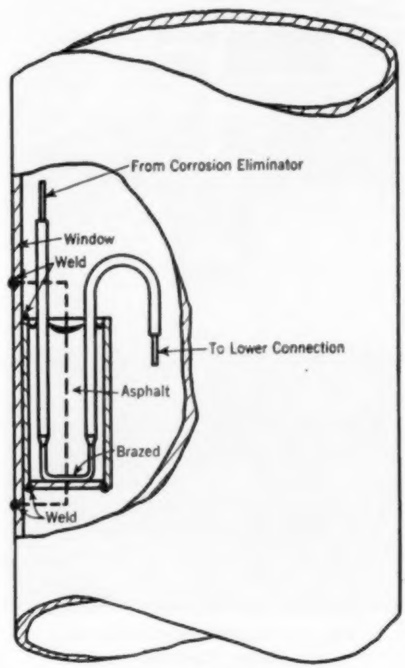

Fig. 14. Detail of Internal Cable Connection

The window or outer portion of the anode pipe at the cable connection is covered with a coat of asphalt to prevent the excessive soil corrosion which would take place directly opposite the connection.

In determination of the placement of vertical anodes in a bed to be installed in soil having a specific resist-

ance of 1,000 ohms per cu cm, the values listed in the example of the horizontal anode apply. On the assumption that the length of each vertical 12-in. anode is 25 ft, reference to Fig. 12 will show that one 25-ft pipe in the given soil will have a resistance of 2.0 ohms. Thus, the number of

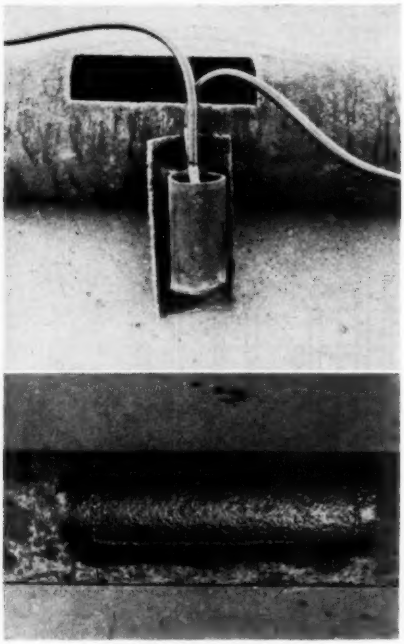

Fig. 15. Asphalting of Vertical Steel Electrode

Top: *Filling cup welded to electrode inside surface.* Bottom: *Asphalt coating in place to protect pipe at connection.*

vertical anodes of 12-in. pipe required to obtain a 0.06-ohm anode bed would be $\frac{2.0}{0.06} = 33$, and the spacing of vertical anodes would be 200 ft.

Graphite Anodes

Graphite anodes are preferred for use in salt water and in soil containing salt, as they have a longer life than carbon under the same conditions. Impregnated graphite backfill is preferred for use in salt water or salt-containing environment as well as in acid environments. It is also preferred if the current density is greater than 0.25 amp per sq ft of anode rod surface. Carbon anodes are recommended for low resistance soils in the absence of salts and are preferred to graphite only when bromides or fluorides are present.

Graphite anodes are available with and without backfill, for current outputs as given in Table 2. When the current discharged from the anode is kept within these recommended limits, the life of the anode rod can be estimated on the basis of $\frac{1}{10}$ lb per amp-yr.

TABLE 2

Current Outputs for Graphite Electrodes

Dimensions *in.*	Weight *lb*	Max. Current Discharge *amp*
2 × 80	20	0.9
4 × 80	25	1.7
6 × 80	130	2.6
With Backfill		
6 × 104	220	3.5
12 × 104	475	7.0
18 × 104	1,100	10.6

A convenient method of installing the backfilled electrodes is to place them in 16-gage steel cans of the dimension given for "backfill" in Table 2, tamping the backfill thoroughly and installing the electrode either vertically or horizontally, as with steel pipe anodes. The anode resistances are calculated with the aid of Fig. 11 and 12, using the outside dimensions of the cans as the anode dimensions.

When these anodes have been in service for some time, the steel cases will be removed by the anodic corrosion, leaving a graphite anode and backfill in contact with the soil. The anode potential drop will then be increased by the back emf of 1 v by the carbon-to-steel backvoltage. When additional area or weight of anode rods is required, the effective length of the rods in horizontal or vertical installations can be obtained as shown in Fig. 11 or 12. When an increased length of horizontal graphite anode is required, a number of units can be placed in a trench and connected to a common bus or cable as shown in Fig. 16.

All carbon and graphite anodes must be vented to permit the escape of the gas formed around them. The gas pockets thus caused reduce the contact area to the adjacent soil.

Fig. 16. Individual Unit Horizontal Electrode Assembly

Magnesium Anodes

Magnesium anodes find their most efficient usefulness in soils ranging in resistance between 100 and 2,000 ohms per cu cm and are economical when the current requirements are low. The effective potential of a magnesium electrode against steel, when stable operating conditions are attained in actual practice, is generally between 0.6 and 0.7 v.

Tests may be made to determine the suitability of magnesium to protect a given steel line, the current required being dependent on the soil resistance and the wrapping resistance on the line. One such test involves measuring the specific resistance of the soil surrounding the pipe and the soil at a location suitable for an anode bed. Another requires installation of a temporary test ground rod, or anode, at the selected anode location. From this ground rod to the pipeline is then sent a current, I, of sufficient strength to provide a given length of line full protection as determined by the McCollum earth current meter indication and the equation: $i = \frac{1{,}000}{r}$, in which r is the soil resistance in ohms per cu cm.

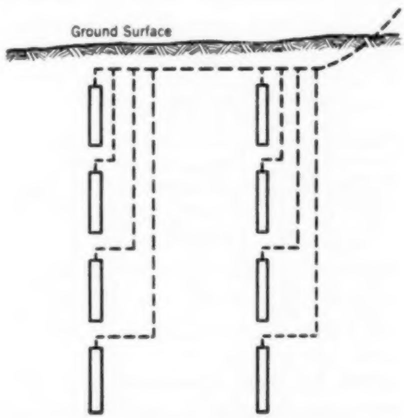

Fig. 17. Individual Unit Vertical Electrode Assembly

When a magnesium galvanic anode is used to protect a steel pipe, the overall potential is limited to a maximum value of approximately 0.6 v when the required current I_0 is passing from the anode through the soil to the pipe.

The difference between this available potential, $E_0 = 0.6$ v, and the net pipe-to-soil potential E_p,* will give the

* Net pipe to soil potential = pipe to copper sulfate electrode = potential minus 0.6 volt (assumed copper to steel potential).

remaining potential E_a which is available to send the current I_0 from the magnesium anode ground bed to the pipe. The anode bed resistance R_a can be calculated by: $R_a = \frac{E_a}{I}$.

The resistance R_1 of a single galvanic anode in a soil of known resistance may be determined by Fig. 11 and 12, using the outside dimensions of the galvanic anode and backfill as the electrode dimension. The number n of galvanic anodes required at a spacing of 150 diameters or more, in order to obtain a ground bed of resistance R_a, can be calculated by: $n = \frac{R_1}{R_a}$. When required by local conditions, the electrode can be placed in horizontal or vertical groups as shown in Fig. 16 and 17.

TABLE 3

Data on Magnesium Galvanic Anodes Used in Tests

Weight of Anode *lb*	Output Rating for 10-Yr Life *ma*	Overall Dimension of Packaged Unit		Weight of Backfill *lb*
		Diameter *in.*	Length *in.*	
17	90	$6\frac{3}{4}$	25	25–28
34	180	$7\frac{1}{2}$	28	35–40

The characteristics and dimensions of the magnesium galvanic anodes used are given in Table 3.

When longer vertical magnesium anodes are required, a number of units can be assembled as shown in Fig. 17, making it possible to obtain a lower resistance anode bed with a limited ground area.

Choice of Electric Protection

Before the type of protection to be applied is finally selected, following the essential preliminary field test, comparative estimates of the various methods should be made. These estimates take account of the comparative annual cost of installing and maintaining the protection with each type of installation, and will include the following items:

1. Cost of electrical protection equipment, service lines, ground electrodes, right of way
2. Interest on investment
3. Taxes on investment
4. Depreciation charges on equipment
5. Renewal charges on ground electrodes
6. Power costs per year
7. Maintenance charges for upkeep of various parts of system.

The foregoing discussion touches only on some of the more essential portions of a cathodic protection system that must be considered in advance. It is well to obtain the assistance of an experienced engineer to study the plans and supervise the actual installation.

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Discussion

F. E. Dolson

Supt. of Distribution, St. Louis County Water Co., St. Louis, Mo.

The author's presentation covers the subject adequately, is easily understood, and represents considerable effort. Although little new has been added, the paper summarizes existing knowledge on the subject.

The portion on "Protective Methods" tends to oversimplify the problem and may mislead readers into thinking that expensive field work can be reduced simply by calculating current requirements rather than determining them experimentally. Current requirements vary considerably depending upon environmental conditions—for example, Hadley (1) reports that minimum current density of 15×10^{-6} to 50×10^{-6} amp per sq cm (14 to 46 ma per sq ft) is required for full protection in an active microbiological anaerobic environment. This requirement is not unusual if protection is being provided against sulfate-reducing bacteria. In contrast, if such bacteria are present in soil of 2,500 ohms per cm resistance, Schneider's formula indicates that only 0.4 amp per sq ft would be required.

The answer to the current density problem has been sought for some time. In reviewing current literature it is difficult to find specific answers or formulas that do not qualify their statements of minimum current density. The National Assn. of Corrosion Engineers recognized the importance of the current density problem and a few years ago established a Technical Practice Committee charged with the responsibility of establishing current density criteria under different environmental conditions. To date this committee has not issued any concrete information.

From the foregoing statements, it might be inferred that there is disagreement with Schneider's formula. Actually, the formula is probably conservative, usually producing considerably more current than is required for adequate protection. As a guide it may be of some use to the engineer. It is still true, however, that the final answer to the amount of protective current required in a given situation should be determined experimentally, inasmuch as the current requirement varies with the conditions encountered. The formula is not applicable to conditions involving sulfate-producing bacteria.

The portion of the paper devoted to ground bed installation is excellent. The charts and graphs should be a valuable aid to corrosion engineers who usually use charts similar to these when establishing ground bed resistance. Any discrepancies between the measured ground bed resistance and that obtained from charts can easily be corrected by making the proper field adjustments.

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Cathodic Protection of Steel Water Tanks

By C. Kenyon Wells

A paper presented on Oct. 25, 1951, at the California Section Meeting, San Francisco, Calif., by C. Kenyon Wells, Asst. Gen. Mgr. & Asst. Chief Engr., Long Beach Water Dept., Long Beach, Calif.

IN determining the performance value of various protective coatings on steel surfaces, no test is more reliable and conclusive than that of subjecting the coatings to actual service conditions over a long period of time. The Long Beach Water Dept. has had the benefit of such a time test in twelve large, steel water-storage tanks constructed a number of years ago, the first group of six tanks having been built in 1932 and the second group of six in 1936. The tanks are all identical in dimension, 132 ft in diameter, 35 ft in height, and have a capacity of 3.5 mil gal each.

The six tanks erected in 1936 were protected on their interior surfaces with coal-tar enamel, and, today, after fifteen years of service, the coatings are still in excellent condition. Satisfactory experiences such as this one have led to a general acceptance by water works men, particularly those in the West, of the coal-tar enamel coating as the most permanent and satisfactory type of application for the protection of submerged steel surfaces from corrosion. Where failure of coal-tar coating has occurred, the breakdown is invariably traced to faulty application, for the cold-applied primer and the hot-applied enamel must be laid down on a thoroughly cleaned steel surface under exacting conditions, and poor technique in the application can result in defective bond. When properly applied, coal-tar base enamels are impervious to moisture, make a tight bond with the steel surface, and maintain a high electrical resistance.

Experience has demonstrated, however, that there are types of coatings which fail to give this kind of service. In 1932 when the department erected its first group of six tanks, the interior surfaces were coated with a product which was then in common use. The coating consisted of a hot-applied enamel over a cold-applied primer but differed from the modern coal-tar enamel coating in that, instead of being formulated entirely of coal-tar base materials, it had been manufactured by blending asphalts and coal tars. This coating began to show definite evidences of failure after eleven or twelve years of service. Increasing numbers of rust spots were observed on the interior surfaces of these tanks when they were emptied each year or two for cleaning. The rust spots appeared on rivets, bottom sheets, side wall sheets, and columns alike. Careful inspection of the rusting areas revealed a deterioration of the coating in which there appeared to be a gradual loss of bond. Large areas of deteriorated enamel could have been removed easily with a putty knife.

In the early stages, to control the corrosion which was becoming increasingly serious, each area of rust was carefully cleaned and given a cold application of coal-tar coating. This was regarded as only a stopgap measure, and it was realized that definite steps would soon be necessary if the steel investment in the reservoir was to be protected.

A check of costs at that time—1945—disclosed that removal of the old coating by sandblasting and application of a coat of coal-tar primer and a coat of coal-tar enamel would cost approximately 25¢ per sq ft, or approximately $8,000 per tank—a total cost of almost $50,000. As this appeared to be a rather large investment, and as favorable reports had been received on the results of applying cathodic protection to steel tanks, it was decided to investigate this newer method of corrosion control.

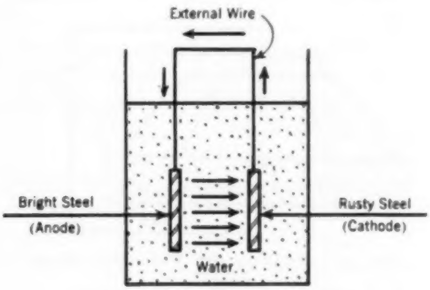

Fig. 1. Corrosion Couple

The basic elements of a corrosion couple are represented by a bright and a rusted piece of steel connected externally and submerged in water.

Corrosion Process

Before discussing cathodic protection, it might be well to review briefly some of the facts concerning the corrosion process. The enormous annual corrosion losses in the water industry are compelling all water works engineers to study the conditions under which corrosion occurs and the methods whereby it may be eliminated, or at least retarded. For this reason, a fairly general working knowledge of the corrosion process exists among these engineers.

It is fairly well known, for example, that if a bright piece of steel and a rusty piece of steel (Fig. 1) are submerged in water and connected with an external wire, a corrosion couple will be formed in which the bright piece of steel becomes an anode and the rusty piece a cathode, resulting in a flow of current through the water from the bright steel anode to the rusty steel cathode. This action is similar to that in a common dry cell (Fig. 2) composed of a zinc anode, a carbon cathode, and the intervening electrolyte. When a resistance load is connected across the two terminals of the cell, current flows through this external circuit from carbon to zinc; but, within the battery itself, current flows

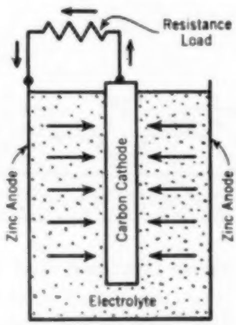

Fig. 2. Dry Cell

Shown are the basic elements of a common dry cell in a simple circuit.

from the zinc anode through the electrolyte to the carbon cathode, completing the circuit. As the current in this cell continues to flow, the zinc anode gradually deteriorates.

With the bright piece of steel acting as an anode to the rusty piece, there is a similar battery action in which the flow of current is from the bright steel anode through the water electrolyte, to the rusty steel cathode, with the circuit completed through the external wire. As the current continues to flow, the anode progressively deteriorates in the same way as the zinc anode did in the dry cell.

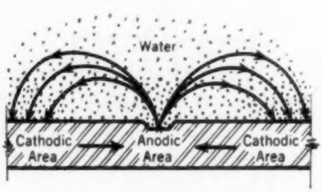

Fig. 3. Freely Corroding Steel Surface

Diagram shows anodic and cathodic areas of a steel surface that has come in contact with water, and the attendant electrochemical corrosive action.

It is generally recognized that the surface of a steel plate in contact with water (Fig. 3) tends to become either anodic or cathodic, depending upon the characteristics of the steel surface at any particular point. A small, bright spot on the steel surface may become anodic to an area covered with mill scale; or imperfections on the steel surface, caused during fabrication, may become anodic to surrounding rusted or tuberculated areas. This condition is like the couple formed by the bright and rusty pieces of steel submerged in water and connected with an external wire, except that these anodic and cathodic areas are in the same piece of steel; and, although the current also flows from the anodic area through the water to the cathodic area, the circuit is completed through the steel plate itself instead of through the external wire.

On the interior surfaces of an unprotected steel water storage tank a great many of these miniature battery cells are formed, each one causing a minute amount of current to flow, and each one causing localized rusting and pitting to occur on the anodic area

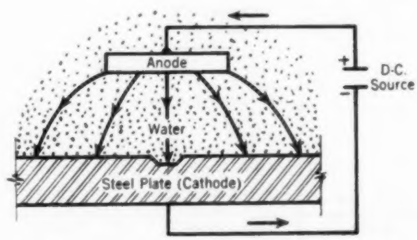

Fig. 4. Cathodically Protected Steel Surface

Method by which anode superimposes a blanket of current on the steel surface, to suppress the current flow accompanying the electrochemical corrosion process, is shown.

where a continuous flow of current is leaving the steel surface. This phenomenon is referred to as the electrochemical corrosion process.

Cathodic Protection

The purpose of cathodic protection (Fig. 4) is to superimpose upon the tank walls a blanket of current which will suppress the current flow accompanying the electrochemical corrosion process. Experience has demonstrated that, when this superimposed current entering the corroding area is strong enough to counterbalance the current

of the many miniature battery cells, corrosion ceases. In other words, the impressed current from the cathodic protection system must be equal to or greater than the current leaving the many corroding, or anodic, areas.

A cathodic protection system for steel tanks consists primarily of a source of direct current, an anode system, and the electrical wiring necessary to connect them. The power unit is ordinarily contained in a steel cabinet mounted on the side of the tank at a convenient height above the ground. It is composed of a multiple-tap transformer to reduce the 110-v alternating current to a more favorable voltage and a selenium rectifier which converts the alternating current to direct current. The anode system consists of a number of anodes suspended in the tank in a carefully designed pattern and connected to the positive terminal of the rectifier. In operation, direct current flows from the rectifier unit to the anodes and then through the water to the metal surfaces of the tank. The circuit is completed by using the tank as a conductor to return the current to the negative terminal of the rectifier.

A correctly designed anode system will provide a uniform distribution of current density to all submerged surfaces of the tank. By the proper placement of the anodes, the electrical resistance of the water between the anode arrangement and the surfaces to be protected can be kept low, thus requiring only a minimum of impressed voltage.

The ideal anode material would be one which resists deterioration while current is flowing from its surface, has a low electrical resistance, is mechanically strong, and is inexpensive. Although no material which fulfills these requirements perfectly is available, the materials more generally used are carbon, graphite, mild steel, stainless steel, aluminum, zinc, magnesium, and platinum.

The total current requirements necessary to achieve complete protection from corrosion must be determined for each specific problem. If the entire tank surface is bare, more current will be required than if a deteriorated coating partially covers and insulates the steel.

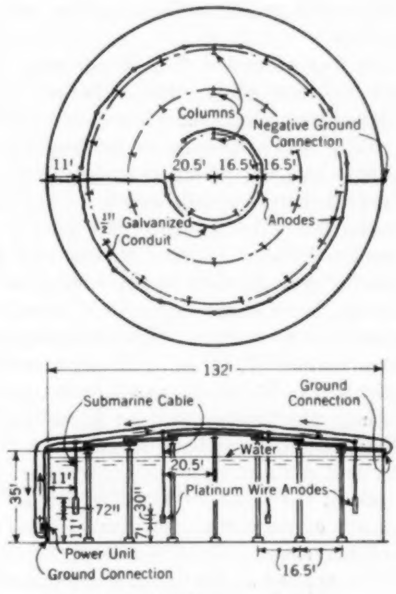

Fig. 5. Location of Anodes

Upper diagram is a plan of the tank, which is shown in deviaton below.

Design of System

After it was decided to explore the possibilities of applying cathodic protection to the six tanks in which the coating was failing, a firm specializing in this type of work was invited to study the problem and submit recom-

mendations. Upon request, they were furnished a drawing of the tank construction (Fig. 5) showing that each tank was 132 ft in diameter and 35 ft high, with a center column and three concentric circles of columns. The concentric circles were 16.5 ft apart and the columns were approximately 10.5 ft center-to-center in the circles. In all, there were 31 steel columns in each tank. A typical sample of the water stored in the tanks was also submitted.

It was estimated that 80 per cent of the interior tank surface below the high-water line remained covered with coating which, because of its insulative value, had the effect of reducing the current-density requirements. On this basis, the cathodic protection firm determined that a current density of 0.3 ma per sq ft applied to the entire submerged area of the tank would provide complete protection. The submerged area totaled 32,000 sq ft, 15 per cent of which was in the 31 roof-support columns, which also required protection. Each column had been formed of a 9-in. and a 10-in. channel placed side to back in the shape of a tee, which formation explains the comparatively large column area. The 0.3 ma per sq ft, when applied to the total area, required a total current of approximately 10 amp. With these data at hand, the consulting firm recommended an impressed operating voltage on the system of 20 v and, accordingly, specified a 24-v power unit designed for a maximum output of 12 amp. It is interesting to note that, with 20 v applied and a current of 10 amp flowing in the system, the indicated resistance of the entire circuit, by Ohm's law, would be 2 ohms.

With these conclusions reached, it remained for the consultants to design an anode system which would distribute a uniform current density of 0.3 ma per sq ft to all submerged surfaces of the tank. The problem was to locate the anodes in such a configuration that, with 20 v applied and 10 amp flowing, the required uniform-current density would result.

Their recommendation, shown in Fig. 5, was to suspend from the roof fifteen anodes of 26-gage platinum wire, 72 in. long, equidistantly spaced on a circle 11 ft in from the side walls of the tank and also located equidistant from the fifteen outer columns. The bottoms of the anodes were to be 11 ft above the tank floor. In addition, there were to be five anodes of 26-gage platinum wire, 30 in. long, suspended from the roof and equidistantly spaced on an inner circle of 20.5-ft radius. These anodes were to be located equidistant from the five inner columns with the bottoms 7 ft above the tank floor. The purpose of these inner anodes was to distribute protective current to the comparatively large center section of the floor area and to the inner columns.

Because the base of each column rests upon the enameled floor and is, therefore, more or less insulated from the steel structure of the tank, it would be necessary to bond each column base individually to the floor with a welded steel cable connection, thus providing a return circuit to the rectifier for the protective current which would flow from the anodes into the columns.

Each platinum wire anode was to be suspended at the correct height from the end of a length of No. 10, 49-strand flexible, plastic-covered submarine cable, the upper end of which was to be secured to the roof deck. The copper conductor of the submarine cable was to be connected to the platinum wire by means of a special plastic connector

which, when made up and screwed together, would provide a watertight connection with low electrical resistance. The lower end of the platinum wire was to be weighted with a ball-type porcelain insulator, the purpose of which was to keep the wire anode stretched in a vertical position. A spun-glass thread secured to the submarine cable above the plastic connector and tied to the insulator was to be adjusted in length until the tension in the platinum wire was relieved. The purpose of this support was to avoid any possibility of breakage of the platinum wire through stress or twisting.

The power unit, enclosed in a steel cabinet, was to be mounted on the side of the tank approximately 5 ft above the ground and connected to the 110-v alternating current supply. The positive lead from the power unit shown in Fig. 5 was to be connected to a No. 10, 19-strand, plastic-covered conductor enclosed in a ½-in., galvanized, rigid conduit and installed up the side of the tank to the roof. From this point, the conduit was to be laid on wood blocking on the composition roof deck in two concentric circles of such radii that the conduit would pass directly over each anode position. At each of these positions, a condulet tee was to be installed with the branch facing down through a hole in the wood roof sheathing through which the submarine cable from the anode would be drawn. The condulet tee was to have a side inspection plate which could be removed and would house the electrical connection between the wire in the conduit and the submarine cable.

The negative lead from the power unit (Fig. 5) was to be grounded to the adjacent tank wall, and, in addition, a No. 10, 19-strand, plastic-covered ground wire was to be drawn through the ½-in. conduit just described in the shortest path to a ground connection on the opposite side of the tank from the rectifier. This second ground connection was to reduce the resistance of the return path for the current flowing into the tank walls on the far side.

To observe the effectiveness of the design before making the installation in all six tanks, the cathodic protection firm was instructed to proceed with the installation of equipment in only one tank. The completed installation produced the anticipated results. With 20 v impressed on the system, a current flow of 10 amp was indicated. The equipment was placed in continuous operation in August 1945.

Of interest is the extreme sensitivity of the system to changes in anode position. Experimentation by the consultants before the final anode design was adopted showed that platinum wire anodes 30 in. long instead of 72 in., placed in an outer circle 13.5 ft in from the side walls of the tank instead of 11 ft, and positioned 7 ft above the floor instead of 11 ft, resulted in a current flow of only 5 amp instead of 10 amp.

Results and Costs

After cathodic protection had been in full operation for seven months, the tank was emptied and inspected. The results were very gratifying. The old rust on the areas where the coating had disappeared had been raised from the surface of the steel, remaining there in a delicate, reddish-brown, honeycomb structure. Beneath the rust, a white, chalky film had formed on the bare steel. The loosened rust was easily brushed away. In fact, in some areas, all traces of it had disappeared. The chalky film, which was mainly calcium carbonate from the water, was being plated out on the submerged surface of the steel. On all spots observed, clean

gray steel was exposed when the white deposit was scraped off. A careful inspection of the entire steel area of the tank below the average high-water line disclosed no new rust areas, and it was felt that the installation was giving full protection. Each succeeding inspection has substantiated this belief. The white protective film has gradually increased in thickness. On the basis of the performance of the equipment in this tank, the five remaining tanks with defective coatings were also subsequently protected.

The cost of the cathodic protection equipment complete and installed was \$1,460 per tank in 1945. The system's life should be not less than twenty years, although it may occasionally be necessary to replace a platinum wire anode. Because platinum does not deteriorate as do most of the anode materials, the only reason for replacement would be accidental breakage of the fine wire. In 1945 the cost of 26-gage platinum wire, 99.5 per cent pure, was approximately \$2.75 per ft; however, its price today is approximately \$4.00 per ft. Each tank required approximately 111 ft of platinum wire, representing a cost of \$305 in 1945 and \$444 at present-day costs. Salvaged lengths of platinum wire can readily be melted and redrawn at a nominal labor cost.

Each cathodic protection unit consumes approximately 400 w of power or 288 kw hr per month. At the rate the department pays for power, this amount of energy would cost approximately \$2.16 per month or \$26.00 per year per tank.

Conclusions

The foregoing discussion clearly indicates that cathodic protection of the interior surfaces of steel water storage tanks is limited to those areas below the average high-water line, and it still remains necessary to protect those steel areas above this level with some type of coating. In sections of the country where ice forms on the surface of the water, the use of cathodic protection is necessarily limited to the non-ice-forming months of the year, because the anode system would be damaged by the rising and falling ice layer. Notwithstanding these limitations, cathodic protection can be utilized to save the water works operator untold thousands of dollars in maintenance costs. After six years of experience with this type of corrosion control, the Long Beach Water Dept. has concluded that cathodic protection is a thoroughly practical method of protecting the submerged steel surfaces of water storage tanks.

The Tucumcari Tank Failure

New Mexico Society of Professional Engineers

A report on the cause of failure of a municipal water tank in Tucumcari, N.M., on Dec. 13, 1951, submitted to J. A. Fleming, City Mgr. of Tucumcari, by the New Mexico Society of Professional Engrs., Albuquerque, N.M.

Editor's Note: On December 13, 1951, Tucumcari, N.M., suffered a disaster when a steel ground-storage water tank failed. At first it was felt the failure resulted from sabotage, from a meteor striking the tank, or from an earthquake. None of these theories was confirmed. A special task group from the New Mexico Society of Professional Engineers investigated and reported that the failure derived from improper welding when the tank was erected. Corrosion to an extent great enough to contribute to the collapse was not evident. The report of the engineering group is published herewith, with discussions by H. O. Hill, Chairman of the AWWA–AWS–NEWWA Committee on Elevated Tanks, and by S. A. Greenberg, Technical Secretary of the American Welding Society.

The tank was first erected and used by the Bureau of Reclamation when the Conchas Dam was being built. The tank was not built according to AWWA specifications—it was of lighter weight steel, and was designed for oil storage. After the Conchas Dam was completed in 1938, the tank was torn down and approximately two years later was re-erected as a water storage tank in Tucumcari.

From the standpoint both of original design and of welding procedures used when it was re-erected, the tank should not have remained intact after it was filled with water. That it did stand and serve for approximately eleven years is evidence of the adequacy of the safety factors involved in tank design. That it failed with a resultant loss of life is evidence that the safety factors were abused.

This disaster illustrates two fundamental rules: [1] a steel tank built in accordance with oil industry specifications should not be used for water storage; and [2] welding is a job for trained men.

In the early morning of Dec. 13, 1951, a steel water storage tank forming a part of the municipal water supply system of the city of Tucumcari, N.M., failed. Several lives were lost, a considerable amount of property damage was inflicted, and the city's water supply was seriously impaired.

Members of the New Mexico Society of Professional Engineers learned of these facts through the public press and radio. They also learned that there existed in the minds of the municipal authorities at Tucumcari, and the public's mind, a considerable question as to the cause of failure of the tank. There were persistent rumors that an explosion had been the immediate cause of failure.

The society, prompted by a desire to render a public service in time of disaster, offered its services to the city of Tucumcari. Tucumcari accepted this offer, and asked that the society send engineers to examine the remains of the tank and report their findings as

to the cause of failure. The society undertook this assignment as a voluntary public service, without charge or cost to the city of Tucumcari. The society, in addition to three of its engineering members, enlisted the voluntary assistance of Arthur P. Bailey, an expert in the theory and practice of welding. This group of four men spent approximately five hours in an examination of the remains of the tank and the wreckage in the immediate vicinity thereof, on Sunday, Dec. 16, 1951.

Scope of the Report

The scope of this report is confined strictly to a determination of the cause of failure of the tank, as evidenced by the physical examination of its remains and the surrounding wreckage. Collateral evidence, such as eyewitness reports or the reports of those on the ground immediately after failure, together with the technical or personal opinions of others than the undersigned, have received no consideration whatever in arriving at the conclusions reached. This report does not concern itself with the extent of damage to property other than the tank itself, and any such damage is referred to only as it may have a bearing on the cause or mode of failure of the tank.

The Tank Before Failure

The tank was located within the western portion of the corporate limits of the city of Tucumcari. It was a part of the municipal water supply system, and is reported by city officials to have been practically full at the time of failure.

It was a circular cylindrical tank 115 ft in diameter and 30 ft high. The bottom was made from steel plates, lap welded, and resting on a cushion of loose pea gravel, which in turn rested, apparently, directly on the soil. The shell was constructed from steel plates $\frac{1}{2}$-in. butt welded in the lower section, $\frac{3}{8}$-in. lap welded in the middle section, and $\frac{1}{4}$-in. lap welded in the upper section. The bottom edge of the $\frac{1}{2}$-in. shell was T-welded to the $\frac{3}{16}$-in. bottom plates by fillet welds, with three passes on the outside and one on the inside. The $\frac{3}{16}$-in. bottom plates extended approximately 3 in. beyond the outside of the shell. The roof was constructed of various thicknesses of lap-welded steel plates, the predominant thicknesses appearing to be $\frac{1}{8}$ to $\frac{1}{4}$ in. The roof was welded to the top edge of the shell. The roof was supported by various types and sizes of steel columns and intermediate "truss" supports, the columns extending to the floor, and being welded thereto. Details of the roof and its mode of support are unimportant, inasmuch as the collapse of the roof was incidental to the failure of the tank. The roof was pierced by a 6-in. iron vent pipe at its center, and by a 30×30-in. square steel door, loosely hinged and not locked, at a point on the circumference. A steel ladder, welded to the outside of the shell, gave access to this door in the roof. There was also an 8-in. overflow hole at the top of the shell.

During the brief period of inspection it was impossible to determine accurate details of exactly what function the tank performed as an integral part of the distribution system—that is, whence it was fed, whither it discharged, etc. These facts seemed relatively unimportant, and were dismissed. The tank appeared to have had a 6-in. inlet rising vertically through the stored water and discharging above its surface, and a funnel-shaped outlet standing perhaps 18 in. above the tank's bottom; the mouth of the funnel was 22 in. in diameter, and the throat perhaps 14 in.

The surfaces of the steel plates forming the shell, both interior and exterior, were in good condition without evidence of serious deterioration by reason of rusting or corrosion. The visible (interior) surface of the bottom of the tank was somewhat more rusted and pitted, but cannot be said to have been in a seriously deteriorated condition.

City officials report that on the day before failure a casual examination of the tank revealed no visible leaks.

The Tank After Failure

The accompanying plat (Fig. 1), made on the ground by Marvin C. May on Dec. 16, 1951, shows the position of the component parts of the tank after failure.

Mode of Failure

None of the undersigned saw the tank fail. The mode of failure, therefore, may not properly be stated as fact in this report. The following statement as to mode of failure represents the professional opinion of the undersigned based on careful examination, the application of fundamental engineering principles, and engineering judgment and experience.

A vertical butt weld in the $\frac{1}{2}$-in. shell near the bottom of the tank failed in tension. The location of this vertical weld was within a few inches of the point where Specimen 7 was taken, as shown on the accompanying plat. It is probable that the sudden snapping of the joint caused a concussion which set up a shock wave of considerable magnitude within the water in the tank, and greatly accelerated progressive failure.

Almost instantaneously, or within the space of a very few seconds, failure proceeded as follows: the $\frac{1}{2}$-in. butt weld in the shell failed throughout its vertical 7-ft length. The two parts of the shell near the bottom, thus unsupported by the tension bond between them, began to move outward under the hydrostatic pressure of approximately 30 ft of water. The only significant force opposing this motion was occasioned by the fillet weld bond between the shell and the $\frac{3}{16}$-in. bottom plates, or floor of the tank. It is noteworthy that the fillet weld did not fail. The $\frac{3}{16}$-in. plates forming the bottom of the tank, however, were unable to withstand the forces tending to move the bottom portion of the shell outward. These $\frac{3}{16}$-in. plates, therefore, began to tear apart along the toe of the fillet weld. This tearing proceeded throughout the circumference of the tank. Simultaneously with these developments, failure of the $\frac{1}{2}$-in. vertical butt weld reached a point approximately 7 ft above the ground, where it encountered the $\frac{3}{8}$-in. plate in the next (upward) section of the shell. The vertical joints in the shell were staggered, and it is noteworthy that the horizontal weld between the $\frac{1}{2}$- and $\frac{3}{8}$-in. portions of the shell did not fail, but rather, the $\frac{3}{8}$-in. plate simply tore apart under the ripping action occasioned by the failure of the $\frac{1}{2}$-in. vertical weld. This ripping of the unwelded portion of the plates in the shell proceeded throughout the vertical width of the $\frac{3}{8}$- and $\frac{1}{4}$-in. plates to the top of the shell. Here the outward motion of the two portions of the now completely separated shell was resisted only by the (quite properly) unsubstantial attachment between shell and roof. The shell tore loose from the roof on both sides of the vertical failure without much resistance, and was now almost completely free to whip outward, tearing the tank's bottom along the toe of the fillet weld, and breaking the bond between shell and roof, as it went.

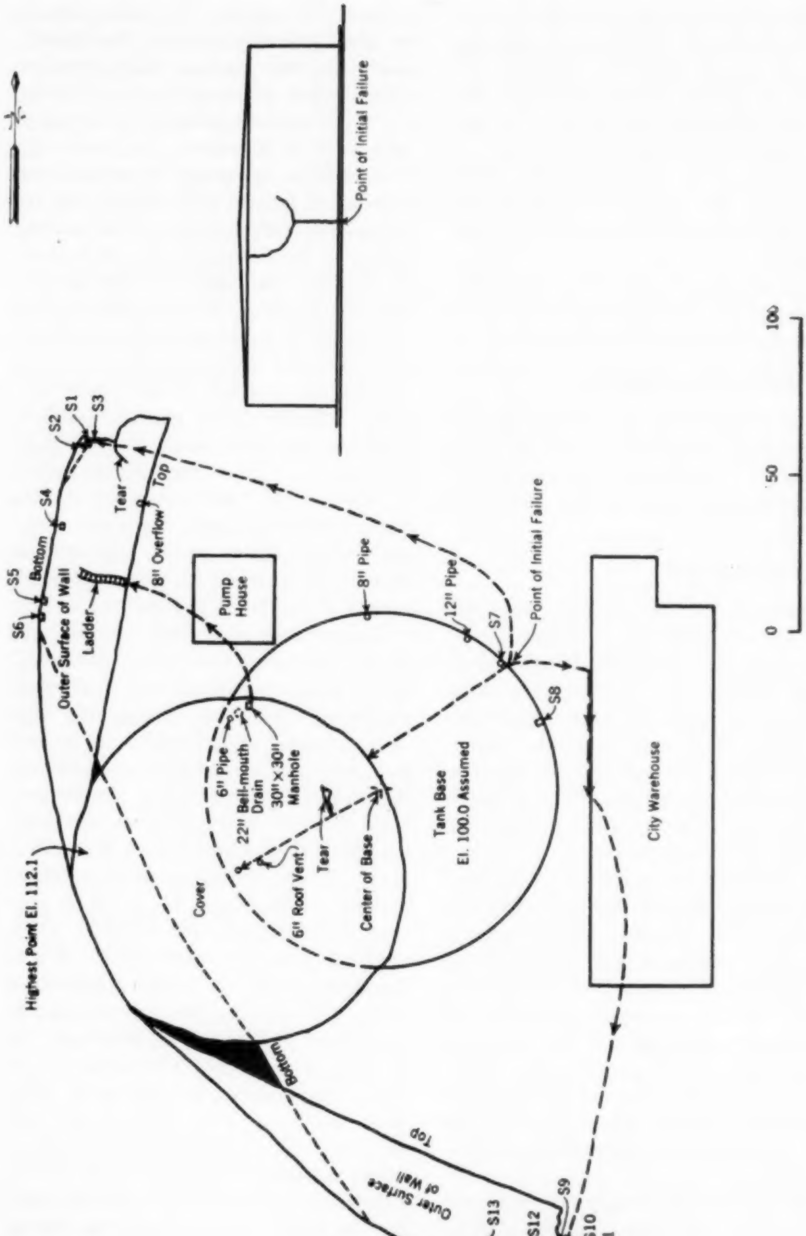

Fig. 1. Plat Showing Position of Component Parts of Tank After Failure

So suddenly did these events occur, that by the time the shell had torn loose from top and bottom almost completely around its entire circumference, there still remained sufficient hydrostatic pressure in a westerly direction to move the shell and roof to the position shown in the accompanying plat. The light roof, offering no resistance to westward motion other than its inertia, remained attached to the shell for the last few feet of its circumference, as shown in the plat. The floor, on the other hand, still being loaded with water, and resting on a rough gravel cushion, was held in place by frictional forces, and became completely detached from the shell.

The almost instantaneous release of the (approximately) 2,300,000 gal in the tank sent a wall of water easterly and northerly, carrying away the warehouse and damaging other structures in those directions. It is probable that the pump house and other structures to the westward and southward suffered considerable damage as a result of being struck by the shell of the tank as it whipped outward or was forced westward.

Design

Thickness of the shell plates indicated that the tank was properly designed with a working stress of approximately 18,000 psi when full.

Cause of Failure

After diligent search, the undersigned found no evidence of an explosion, nor of any forces having been exerted on the tank other than those which it would normally have been designed and constructed to withstand.

It is the considered professional opinion of the undersigned that the tank failed because of a poorly welded vertical butt joint between the $\frac{1}{2}$-in. plates of the shell in the manner previously described.

Because the joint failed throughout its entire vertical length, it was impossible to test any part of it in tension. Careful visual examination of both halves of this failed joint, however, revealed the following facts which, together with tests hereinafter mentioned, are here set down in support of the opinion of the undersigned as to the immediate cause of failure:

1. Prior to welding, the abutting edges of the two $\frac{1}{2}$-in. plates had been flame-cut. (This is reported to have occurred during the dismantling of the tank at a previous location before it was acquired by the city of Tucumcari.)

2. Although $\frac{1}{2}$ in. in thickness, neither plate had received any edge preparation whatever. Thus, instead of employing one of the conventional V, J, or U grooves which are required by good practice in butt welding plates as thick as $\frac{1}{2}$ in., the welder had simply constructed a square-groove, double-welded butt joint.

3. Failure to provide necessary edge preparation resulted in the groove being only partially filled with filler metal. The blackened edges of the abutting parent metal, where the filler metal had not completely filled the groove, bore the unmistakable marks of the cutting flame previously mentioned. The average depth of the groove (throughout the 7-ft length of the joint), which had been filled with welding metal, is estimated at less than $\frac{1}{10}$ in. on each side of the double weld. Thus, approximately 30 per cent of the groove had been filled. At some points the amount of filler which had entered the groove was practically nil, and the plates were apparently being held together by bead reinforcement applied outside the $\frac{1}{2}$-in. thickness of the plates.

4. Although cross sections of the failed joint were not etched, it may reasonably be assumed that penetration was inadequate, in view of subsequent comments herein concerning those joints which were etched.

5. The conditions above described prevailed in varying degrees throughout the 7-ft length of the joint, measured along its axis.

Specimens and Tests

Specimens were flame cut from the tank on Dec. 16 under the supervision of Arthur P. Bailey. The locations of the several specimens are shown on the accompanying plat.

A certificate of tests * was supplied to the society without charge by the Albuquerque Testing Laboratories. The following comments are set forth as a supplement to the certified test results.

Tension tests were made on Specimens 4, 5, 6, and 13. Specimens 4, 6, and 13 were ½-in. double-welded butt joints; No. 5 was all parent metal. All tested specimens were machined down to 1½-in. width, thus producing a nominal cross-sectional area of ¾ sq in. The bead reinforcement was ground from Specimens 4 and 6, bringing the thickness of the weld down to the ½-in. thickness of the plates. The plates in Specimen 13 were slightly offset—that is, they did not lie in the same plane. It was therefore impossible to grind off the reinforcement from this specimen without damaging the plates. There was very little reinforcement on this specimen, however, and the joint had something of the appearance of a double fillet weld, although the plates were not lapped, but merely slightly offset.

Specimen 5, which consisted of all parent metal, broke at a tensile stress of 61,826 psi, indicating that the plate was sound. Specimens 4 and 6 broke in the weld at 61 and 66 per cent, respectively, of the breaking strength of the plate in Specimen 5. Specimen 13 (offset) broke in the plate, not in the weld, at 95 per cent of the breaking strength of Specimen 5.

The appearance of the broken cross sections of Specimens 4 and 6 was that of a "sandwich" composed of two outer layers of weld metal encasing an inner layer of slag. It was estimated by visual inspection that the three layers were approximately the same thickness. Thus, about two thirds of the joint appeared to be weld metal, and one third slag.

Prior to breaking, Specimens 4 and 6 were etched on their edges. It was obvious from the pattern of the etching that these plates had been given single groove edge preparation before welding, in accordance with standard welding practice. It was equally obvious from the sharp, straight line of demarcation between the welding metal and the parent metal, that welding techniques employed in constructing these joints had not produced any perceptible penetration.

Conclusion

In conclusion, the undersigned wish to express to the city of Tucumcari, and especially to City Manager Fleming, their appreciation of the hospitable, helpful, and wholly unprejudiced treatment accorded to the Society during the inspection.

THE NEW MEXICO SOCIETY OF PROFESSIONAL ENGINEERS
CHARLES E. BARNHART, *President*
MARVIN C. MAY, *Member*
D. R. W. WAGER-SMITH, *Member*
ARTHUR P. BAILEY, *Associate*

* The certificate accompanied the presentation of the original report but is not reproduced here because of space limitations—ED.

Discussion

H. O. Hill

Chairman, AWWA-AWS-NEWWA Committee on Elevated Tanks, Bethlehem, Pa.

If the tank had been constructed in accordance with AWWA D100-48, Standard Specifications for Elevated Steel Water Tanks, Standpipes, and Reservoirs, the shell plate thickness in the lower course of the tank would have been 0.7 in. thick instead of 0.5 in. In addition, Specification D100 requires that the welding procedure and the welding operators be qualified. This qualification, with proper supervision, would have eliminated the very faulty plate welding edge preparation. Also, the inspection requirements of the specifications require the removal of test plugs from the welding; such removal would have indicated that inadequate welding was being performed.

S. A. Greenberg

Technical Secretary, American Welding Society, New York, N.Y.

To this writer, the water tank failure at Tucumcari, N.M., emphasizes the importance of our present standards for welding in water works construction (and all other types) and the need for continued promotion of these standards for use by competent engineers and inspectors.

The immediate cause of tank failure, as reported in an investigation by the New Mexico Society of Professional Engineers, was the failure of a groove weld in a vertical seam in one of the bottom courses of the tank shell. Examination showed that this weld was "a 'sandwich' composed of two outer layers of weld metal encasing an inner layer of slag," the inner layer of slag being one third the total plate thickness.

Had the standard specifications for steel tanks of the American Water Works Assn., the American Welding Soc., and the New England Water Works Assn. been followed, this weld would have been designed and made with 100 per cent penetration. The report of the New Mexico Society of Professional Engineers also indicates that plate edge preparation did not conform to the standard specifications.

Nothing is said concerning construction practices used in erecting the tank. Although it is a matter of conjecture, it seems safe to presume that the requirements of the specifications for qualification of welders, welding procedures, and removal of test plugs for inspection of weld quality were not followed. Conformity to these requirements would have indicated the inadequacy of the welding used, as did the tension tests made during the investigation *after failure*.

Publication of the report on the Tucumcari incident in the JOURNAL, pointing out the nonconformance to the specifications, should emphasize the value of these specifications and prevent construction that may result in the recurrence of such an unfortunate, and avoidable, loss.

Proportioning Cylindrical Reinforced Concrete Tanks for Minimum Steel Content

By E. B. Meier

A contribution to the Journal by E. B. Meier, Asst. Professor of Civ. Eng., Univ. of Nebraska, Lincoln, Neb.

THE basic economic problem confronting the structural designer for many years has been to proportion a required structure for minimum money expenditure. With the current rise in military requirements for the metals used in construction, and with no foreseeable reduction in defense allocations, it appears that the structural engineer may be confronted for a number of years with a second basic economic consideration—that of minimum use of critical steel. Such a policy may become a patriotic responsibility as the natural resources of the nation are used increasingly and at sustained high levels. These two economic considerations do not necessarily occur simultaneously.

One quantity use of steel is the reinforcement of covered cylindrical concrete tanks or basins. Ordinarily, the design of a cylindrical reinforced concrete tank is the same whether the tank is aboveground, embedded, embanked, or unembanked. The quantities of concrete and steel required, then, are independent of the level of the surrounding earth. Thus, unembanked, aboveground cylindrical basins are ideal for investigating proper proportioning for economic use of money and materials. A further consideration in the economic design of cylindrical reservoirs is the possibility of using prestressed walls and dome rings.

Comparisons of prestressed and conventional designs are presented in this report in a range of volumes from 200,000 to 6,000,000 gal, representing water storage capacities sufficient for population segments from approximately 2,000 to 50,000 persons.

For a given capacity, the volume of material in the walls of a cylindrical tank will vary approximately as the square root of the height of the tank. The volume of material in the base of such a structure, assuming constant floor thickness, will vary inversely with the height. The volume of material in the roof or cover, even for domes, will vary approximately inversely with the height. Obviously, for large heights the wall quantities will dominate, whereas for small heights the cover and base quantities will prevail. Thus, total tank cost, for a single capacity, will be influenced principally by wall costs when the diameter is small, and by dome and floor costs when the diameter is large.

Cost comparisons were based upon unit costs prevailing during 1950–51. Total costs and cost ratios were computed by applying unit costs to wall, ring, cover, and base quantities, respectively. The floor thickness used in all computations was 6 in.

For all designs, spherical domes and rings were compared with elliptical domes, and the method of cover which

required minimum quantities of concrete and steel was used. On this basis, it was indicated that all capacities above 2 to 3 mil gal should be covered with elliptical domes for conventional designs only. All other capacities for conventional and prestressed designs should employ spherical domes and rings.

Unit Stresses

Conventional designs with the following stress values were investigated: f_s (allowable unit working stress in steel)—14,000 psi, f_t (allowable unit tensile stress in concrete)—300 psi, and n (ratio of moduli of elasticity, steel to concrete)—10.

Prestressed designs studied had the following stress values: f_s—100,000 psi, f_{si} (initial prestress in steel—tension)—140,000 psi, f_{ci} (initial prestress in concrete—compression)—1,000 psi, and n—9. These stress values were used because the investigations of others (1) have shown that they permit practicable steel placement for large wall heights within the range of capacities considered.

Minimum Steel and Concrete

Conventional and prestressed designs were investigated in 0.2-mil gal increments to 1.0 mil gal, then in 1.0-mil gal increments to 6.0 mil gal. For each of these capacities, concrete and steel quantities were determined for heights up to 52.5 ft in increments of 7.5 ft. These quantities were plotted against height of tank for each of the capacities, and smooth curves were drawn through the points. Each of the curves revealed a minimum within the range of heights investigated. These minimum points provided the data for the curves in Fig. 1.

Figure 1 shows that in order to accomplish proportioning for minimum quantities of concrete when employing

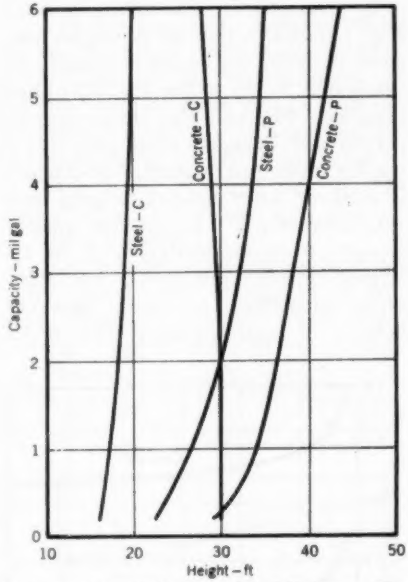

Fig. 1. Proportioning for Minimum Use of Materials

Each of the curves represents the height for various capacities that will use minimum amounts of steel or concrete, in accordance with the following key:

Steel-C—Minimum Steel, Conventional Design

Concrete-C—Minimum Concrete, Conventional Design

Steel-P—Minimum Steel, Prestressed Design

Concrete-P—Minimum Concrete, Prestressed Design

conventional design, a relatively constant height ranging from 30 ft for the smaller capacities to 28 ft for the larger capacities should be used. There is a slight reduction in ideal height for minimum concrete volume in the larger capacities because wall quantities begin to predominate at lower heights at which the capacity increases as a result of wall thickening to accommodate the greater quantities of steel required. For given heights, diameters increase as the capacity is increased, thus explaining the greater need for steel.

In proportioning for minimum steel, Fig. 1 indicates that ideal heights vary from 16 ft for the smallest capacity investigated to 20 ft for the 6.0-mil gal capacity. Figure 1 also shows that for prestressed designs, ideal proportioning for minimum quantities of concrete is achieved when heights ranging from approximately 29 ft for the smallest capacity to approximately 44 ft for the largest capacity are used. For minimum quantities of steel, however, heights of approximately 22.5 ft for the smallest tanks to 45 ft for the largest tanks should be employed.

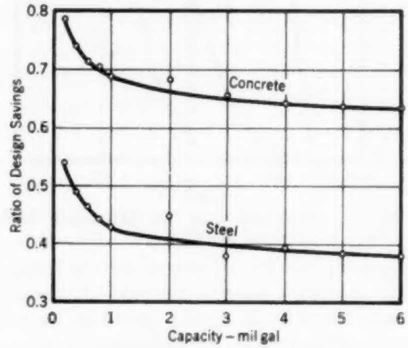

Fig. 2. Proportionate Saving of Materials in Prestressed Designs

The concrete and steel used in prestressed design is expressed, for tanks of various capacities, as a fraction of that used in conventional design.

The practical use of the curves in Fig. 1 merits attention. The contractor who bids reinforced concrete on unit costs which were not derived through separation of concrete and reinforcing steel quantities will not present a bid which will be most advantageous to the owner, assuming the basin was proportioned for minimum steel. The second basic economic consideration, however—minimum use of critical steel—will be attained, as the steel actually used in the project during construction will be the minimum for that particular capacity.

Quantity Comparison

Using ideal proportioning for minimum quantities of concrete and steel, the ratios of the prestressed design quantities to the conventional design quantities were computed and plotted in Fig. 2. Expressed as a percentage, the prestressed design requires 78 per cent as much concrete for the smallest capacity as the conventional design, and 63.5 per cent as much for the largest capacity. There is a greater saving in steel, the amount required by prestressed designs ranging from 54 per cent as much in the smallest capacity to 38 per cent as much in the largest.

Cost Comparison

Figure 3 compares costs of prestressed basins with conventional basins when ideal proportioning for minimum steel is used. The unit costs of the dome, base, conventional wall, and conventional ring are representative of average low-bid values for 1950–51. The range of unit costs for prestressed walls and rings adequately covers the variations in these prices for the same period, 1950–51. If the unit cost of prestressed walls and rings is less than approximately 1.7 times the unit cost of conventional walls and rings, a definite cost advantage can be realized using prestressed design. As long as the ratios between the unit costs are approximately the same as those used in Fig. 3, the percentage relations will remain about the same, regardless of the numerical values of the unit prices used in such a comparison.

Using minimum steel proportioning and unit costs, Fig. 4 was prepared for conventional design. This curve clearly indicates that after a capacity

of approximately 1.0 mil gal is exceeded, the cost per unit volume of cylindrical reinforced concrete tanks increases at a decreasing rate as the capacity is increased. The variation ranges from $0.028 per gal for capacities of 1.0 mil gal or less, to $0.033 per gal for a capacity of 6.0 mil gal, an increase of almost 18 per cent.

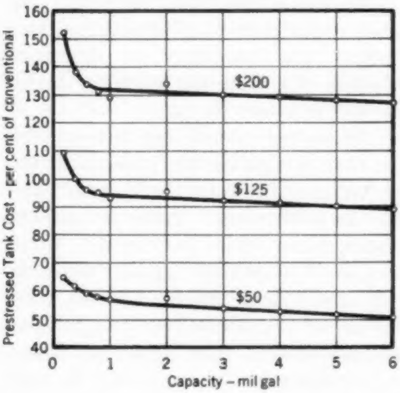

Fig. 3. Total Cost of Prestressed Tanks

The total cost of prestressed tanks is expressed as a proportion of the total cost of conventional tanks. Three different cost rates—$50, $125, and $200 per cu yd—are given for the cost of constructing prestressed walls and rings. Other costs per cu yd were considered stable: bases costing $35; domes, $75; and conventional walls and rings, $75.

Conclusions

1. Although minimum costs would probably be realized by proportioning for minimum concrete, the patriotic conservation of steel may be achieved by proportioning for minimum steel.

2. The ideal heights of cylindrical reinforced concrete tanks range between 16 and 45 ft depending upon whether minimum concrete or minimum steel is desired, and whether conventional or prestressed design is employed.

3. For the range of capacities considered, the prestressed design invariably yielded savings in quantities of concrete, ranging from 22 per cent in the smallest capacity to 36.5 per cent in the largest capacity; and in steel, varying between 46 per cent less in the smallest capacity to 62 per cent less in the largest capacity.

4. For volumes up to approximately 1.0 mil gal, the cost per gallon of capacity remains constant. For conventional designs, the cost per gallon then increases rapidly between 1.0 and 3.0 mil gal and increases diminishingly to 6.0 mil gal. The increase in cost per gallon between 0.2 and 6.0 mil gal is approximately 18 per cent.

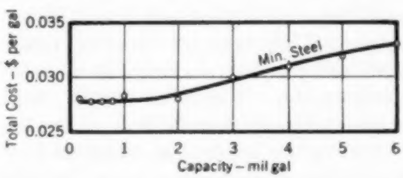

Fig. 4. Increase in Unit Cost of Conventional Tanks With Size

Conventional tanks, if proportioned for minimum use of steel, increase in cost per unit of capacity as the capacity increases. Walls, rings and domes were computed at $75 per cu yd; bases at $35.

Acknowledgment

The author wishes to acknowledge the practical comments made by V. R. Kneer in discussing this paper, and to state that more time and more experience with prestressed designs will be gained only if the subject is kept open for discussion.

References

1. Ernst, G. C.; Brunken, C. O.; & Riveland, A. R. Relative Economy of Prestressed and Conventional Reinforced Concrete Reservoirs. Jour. Am. Concrete Inst., **23**:6:505 (Feb. 1952).

Discussion

V. R. Kneer

Engr., Alvord, Burdick, & Howson, Chicago, Ill.

The author's basic conclusion that the ideal wall height ranges between 16 and 20 ft confirms the writer's long-standing opinion. This range also applies to square or rectangular basins. Most of the writer's designs range from 15 to 20 ft unless variations are dictated by other considerations.

If savings are considered solely from the standpoint of concrete quantities, higher wall heights—up to 44 ft—could result in economy, as concluded by the author. Unit cost for erecting formwork and placing concrete will increase considerably, however, with wall heights of more than 20 ft.

The author selected an unembanked, aboveground cylindrical tank for the analysis. It has been the writer's experience that it is good economy to protect concrete reservoirs from the elements and from extreme temperature ranges with earth embankment, both top and sides. If that procedure is followed, unit excavation and backfill costs will also increase with increased wall heights. This is especially true if the tank bottoms must be placed below the water table. When in operation, the shallower tanks have the advantage of lower pressure ranges.

The writer feels that more time and more experience with prestressed tank designs are necessary to determine whether the material savings justify the loss in safety factor.

The author's fourth conclusion—that the cost per gallon increases with size for volumes between 1 and 6 mil gal—may be misleading. With the author's assumptions, this conclusion would prevail if quantities, again, were the only cost consideration. Usually there are other considerations which upset this conclusion. The writer has found, on contracts let over a period of more than 30 years for both circular and rectangular reservoirs ranging in size from 0.5 to 30 mil gal, that the larger the job, the lower the cost per gallon.

Remote Controls to Eliminate Surge and Water Hammer in Water Systems

By John S. Shute

A paper presented on May 18, 1951, at the Pacific Northwest Section Meeting, Vancouver, B.C., by John S. Shute, Columbia Water Co., Washougal, Wash.

THE public water supply of the city of Washougal, Wash., is provided by the Columbia Water Co. Like all water utilities, regardless of type of ownership, size of system, or kind of service rendered, this company has found it increasingly difficult to continue performing its function of providing water day and night, winter and summer, at rates which have lagged behind the prices of other commodities. One of the steps the company has taken is the elimination of all expenses which, by any reasonable standard, could be considered unnecessary. And a second, which has been aimed primarily at overcoming many troublesome and, at times, serious interruptions of service, has been the installation of automatic, synchronous alarm and surge controls at the pumping stations.

The capacity of the Washougal water system exceeds 5 mgd. The water is supplied by five deep well turbine pumps, three located at Station No. 1 and the other two at Station No. 2, three miles away. Maximum size of mains is 12 in., and customer services are connected to all mains up to a point within a few hundred feet of each pumping station.

In establishing the control system, several types of pump controls were used, but none of them was very successful. Float switches, pressure gages, and other similar types of controls proved to be inaccurate or otherwise troublesome under certain conditions, either flooding or emptying the tank. Special difficulty was encountered when power failed with two or more pumps in operation. If, when power was restored, an attendant were not present to disconnect the additional pump or pumps that had been in operation, more than one would attempt to start. Under such conditions, unless there is excess capacity in the transformer bank, another power failure will result. Additional difficulties with the first types of controls used were the lack of protection against backspin and the absence of any signal or alarm to give warning if any of the pumps failed to start.

Operational Improvements

To improve these conditions, a pipeline was installed from the riser of the elevated tank and connected to a mercury column in the control room. In the column, contact rods were inserted in place of the pressure switches, float switches, and other such equipment. These contacts operate synchronous, motor-driven relays, one being used for each pump. The control can be

adjusted for any desired water elevation, with the "all off" position only 3 in. from the overflow of the tank. This arrangement provides for maximum use of the tank.

The synchronous relays have several special features. If two or three pumps are operating when a power failure develops, all pumps will drop off the line with the first momentary failure. When power is restored, the first pump will not return to the line for 4 min., this lapse giving the power company's oil switches ample time to restore power or to lock out. After the first pump is in operation, the second one will start within a predetermined interval. If a third relay is required, that pump will start operating in the same manner. Inasmuch as it is impossible for any two pumps to start simultaneously, these relays provide complete protection against catching the pumps in a backspin.

Mercury columns have been used previously in many different ways. Their application in Washougal in conjunction with the motor-driven relays, however, is an example of their versatility in providing flexible operation. As part of the synchronous relay system, an alarm was installed to signal the shop and office, as well as the city fire station, if one of the pump controls indicated a low water level, and the pump did not start. Such a condition may arise when a holding coil burns out or a fuse blows.

Station 2 Development

Pumping Station No. 2 was developed to maintain pressure at the extreme end of the system. The mains were installed, the pumps were set, and two pressure switches were connected to the same type of synchronous control used at Station No. 1. One of these switches was to start the pump at the lower pressure; the other was to stop it at a predetermined high pressure. The switches were used in conjunction with the synchronous motor relays and valves that were expected to help relieve surge and vacuum. When the pump was at a starting pressure of 58 psi, the surge carried the pressure to 125–130 psi in the mains and then settled down to a 61-psi pumping pressure. The same surge developed when either of the pumps stopped. Fortunately, there were no breaks in the mains.

Other difficulties were created by irregular use of the pumps at the new station. Silt or dark water developed in the well and pump column when the pump was idle. Then this was pumped into the system with the first inrush of water as the pump started. Also, air from the pump column entered the main, creating enough air pressure in the piping of homes near the station to knock drinking glasses out of customers' hands when they opened their faucets.

Motor-Operated Valve

To correct these various troubles, a motor-operated valve actuated by the synchronous motor-driven relay was installed. Figure 1 shows the surge before and after the installation of the synchronous motor control and valve. The same basic pump control used to prevent more than one pump from starting at a time was used, but some additional features were included. When the control receives a call for water, instead of starting the pump, it opens a motor-operated bypass valve on a tee inserted between the pump and the main-line check valve. When this valve opens, the pump starts, but the valve is adjusted to allow only the amount of

water to pass through it that is normally pumped into the mains. By this procedure all excess surging of the well is eliminated. Discharge of the first inrush of water to waste rather than to the mains clears all the silt, light sand, and discolored water which are always found in the pump column if the pump has remained inoperative for any length of time. It also expels all air from the pump column. The motor-operated

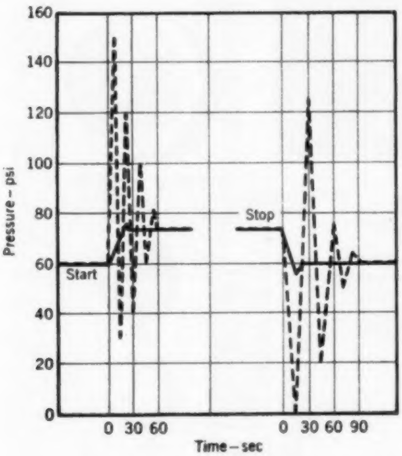

Fig. 1. Surge Elimination

The broken line indicates pressure without the synchronous motor control and surge valve, and the solid line shows the pressure after the control was installed.

valve then slowly closes again, operated by the synchronous, motor-driven relay. With the closing of the bypass, the main-line check valve gradually opens and permits the full amount of water to be pumped into the mains without surge. Using this procedure, when the pump started, trouble from surge, discolored water and silt from the well, and air in the pump column was eliminated.

Started in this way, the pump continues to run until stopped by the controls. Then the operation reverses itself. The motor-operated bypass valve opens slowly and discharges the water outside the pump building until the normal pipeline pressure is reached. While the motor-operated valve is slowly opening, the main-line check valve is slowly closing. A few seconds after the motor-operated valve is completely open, the pump stops. In this operation, the starting surge is reduced from 80 psi to 5 psi, and the corresponding vacuum, or surge, at stopping is practically eliminated. Also, the pressures are checked without use of suppressors on any of the gages.

As a final stage, to ascertain that there is no opening to the wells when the pumps are not running, the synchronous control comes into operation and closes the motor-operated bypass valve, thus preventing any foreign substance from entering the wells.

Tank-Level Gage

It was considered of vital interest to be able at all times to determine the exact water level in the tank without making a trip to check it. The difficulty encountered in using the customary equipment for determining the elevation of water by reading the pressure gage at the office was that, if water demand increased, the gage would give a false reading because of the difference of flow in the mains. To overcome this, a tank water level signal and indicator was installed. Figure 2 shows the panel for the synchronous motor surge control including the tank-level indicator. This signal transmits an electric impulse over a leased telephone circuit which connects Pumping Stations No. 1 and No. 2, the office, the shop, and the city fire station. At each of these locations, an indicator

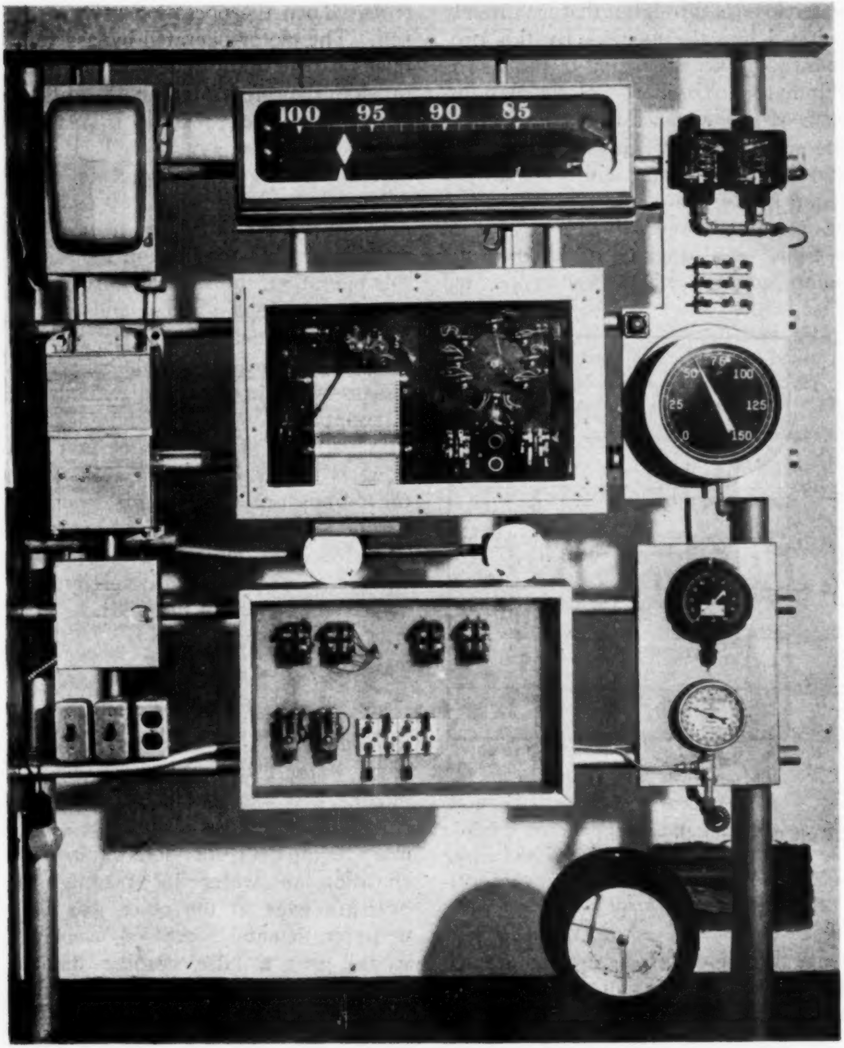

Fig. 2. Panel for Synchronous Motor Surge Control

The large gage at the top is the tank-level indicator which shows the water level in tank 3 miles from pumping station. Below this gage is the synchronous motor control. At the lower right is a recording meter which shows pressure when pumps are standing or operating.

which, at a glance, gives the exact water level in the tank, measured in feet, to an accuracy of 0.25 ft, is installed. This signal and indicator is controlled entirely by the water level in the tank, without regard to flow conditions in the mains. There are no special electronic circuits, standard equipment being used throughout the water level control system.

To insure the accurate operation of this system, it is constructed so that, if any of its working parts or the transmission line fails, or if any kind of electrical trouble is experienced, an alarm is imediately sounded at the office and the city fire station. This alarm is most important, for, without it, the control circuit could be out of order for hours, or even days, before the trouble would be detected.

Pump Operation Signal

One improvement led to another. With the necessity for making trips to the tank virtually eliminated, it appeared desirable, too, to cut down on the visits to pumping stations. To accomplish this, a so-called "telltale" system, which also operates on a telephone circuit, was developed and installed. By dialing an unlisted number assigned to the circuit, the operator receives a signal from the automatic equipment in the pumping station indicating which pumps are running. This information is invaluable, as it enables a check of the pumping stations any time of the day or night without a trip to them.

Acknowledgment

The author wishes to express appreciation to the engineers of the state health department, the Washington Surveying and Rating Bureau, and the Washington Public Service Commission, all of whom were most cooperative in developing the control system described here.

Installation Experience With Pressure Pipe, Joints, and Linings

By Ray A. Jackson

A paper presented on Nov. 1, 1951, at the Chesapeake Section Meeting, Baltimore, Md., by Ray A. Jackson, Asst. Engr., Washington Suburban Sanitary Commission, Hyattsville, Md.

IN the Washington Suburban Sanitary Dist., water mains are laid to grade. Plans for water mains include profiles showing, to tenths of a foot, the ground line and elevations of the invert of the main. These measurements are shown at intervals of 50 ft or less, and also at fittings and valves. Offset grade stakes are set at these stations by a line-and-grade party, and the foreman or the contractor is furnished with a cut sheet to guide the laying of the main. Each pipe is graded with a rod, and aligned in the trench as carefully as possible by eye. A minimum cover of 3½ ft is provided. Earth trenches at all points below the top of the pipe are required to be not wider in the clear, on each side of the outside barrel of the pipe, than the following: 8 in. for diameters to and including 16 in., 10 in. for 18- and 20-in. diameter pipe, 12 in. for 24- and 30-in. diameter pipe, 15 in. for 36- and 42-in. diameter pipe, and 18 in. for 48-in. diameter pipe and larger.

The entire interior surface of the pipe is swabbed with a solution of chloride of lime in the proportion of 3 oz per gal of water, before the pipe is lowered into the trench.

Two alternate methods of refill are provided by new specifications adopted in late August 1951. In paved streets, all refill must be consolidated by mechanical tamping in 6-in. layers. In other than paved streets, the refill may be consolidated by tamping, or by puddling, with the top 2 ft being mechanically tamped in 6-in. layers. Puddling only is required in a right of way. With asbestos-cement pipe the bottom 2 ft must always be tamped in 4-in. layers.

Types of Pipe

Six types of pipe have been used in water main construction. In the early days of the commission, pit-cast iron pipe was used for water mains. Centrifugally cast-iron pipe processes are now used exclusively. Reinforced concrete, steel, and asbestos-cement pipe have also been used. Shortly after the commission was organized, it purchased one of the local water systems containing about 38,600 ft of wood-stave pipe wrapped with wire. The diameter of the pipe varied from 2 to 16 in. Approximately 1,500 ft of wood-stave pipe was also purchased and laid by the commission in a low-pressure line. At present, the wood-stave lines are giving considerable trouble owing to the deterioration of the wire wrappings; these mains are being replaced with cast-iron as rapidly as possible.

The Washington Suburban Sanitary Commission was one of the pioneers in the use of centrifugally cast

pipe. At first, trouble was experienced in cutting the pipe. Saw-toothed cuts and longitudinal cracks developed. After the mains were placed in service, circumferential and longitudinal breaks frequently occurred. These breaks and faulty cuts were caused by poor annealing. Foundry inspections and consultations with metallurgists indicated that there was too long a period —and therefore too much cooling— between the time the pipe was spun and the time it was annealed. The annealing procedure has been greatly improved and has presented no further difficulties.

Jointing Compounds

For cast-iron bell-and-spigot pipe, lead and various brands of sulfur jointing compounds with braided hemp or rubber packing have been used. A joint in which a wedge-shaped or round rubber ring is used in place of the braided hemp was tried experimentally. It was satisfactory on straight runs but the round rings especially had a tendency to kick out when the pipe was crimped to make a short radius curve. Although the rubber ring packing is more sanitary than the braided hemp, it is not used because it is not as satisfactory in its installation. The commission chlorinates all of its mains before use and obtains samples for bacteriological tests. The mains are not put into service until the test indicates that the water in the mains is of satisfactory quality. During hot weather, rechlorination of the main is occasionally necessary. Generally, however, the tests are satisfactory after the first chlorination.

Lead was originally used for bell-and-spigot joints, but at present sulfur jointing compounds are used in all except special circumstances, such as stream crossings, where lead is used because of its ability to be recalked if leaks develop. One disadvantage of sulfur jointing compounds is their inability to be used if overheated. Lead, on the other hand, only oxidizes if overheated, and can be skimmed and used. The high cost of the lead joint, however, compared with that of the sulfur joint, has resulted in the adoption of the latter. The present cost of lead by weight is approximately three times that of the sulfur jointing compound used by the commission. In addition, it takes approximately four times the weight of lead as sulfur jointing compound to make the average joint. Also, it is more costly to calk the lead joint than it is to pour the sulfur jointing compound. The commission has been using sulfur jointing compounds for almost 30 years, and no trouble has developed from sulfur bacteria.

Except on large supply lines, the commission does not require special leakage tests on its new water mains. The mains are charged at the local pressure at the time of chlorination and are then visually observed for leakage.

In 1936, a cast-iron line jointed with sulfur-base compound and consisting of 3,900 ft of 12-in., 5,000 ft of 14-in., and 11,300 ft of 20-in. pipe was laid. The line was tested at pressures between 90 and 120 psi, with a specified allowable leakage of 80 gpd per in. of diameter per mile. Tests were made at various times for a period of three months. The leakage was originally about fifteen times the permissible limit, but after several leaks were repaired and the joints "took up," it dropped to about three times the limit.

At this stage it was necessary to discontinue the tests because of the necessity to place the lines in service.

Also in 1936, approximately 13,900 ft of 12-in., cast-iron main were laid. It was specified that this main was to be tested at line pressure, with an allowable leakage of 80 gpd per in. of diameter per mile. Inasmuch as a connecting line had not been completed, this line could not be tested for approximately two months after completion, but water was pumped into the line and allowed to remain there for that time. When water was available for testing, leakage tests were made on the line during a 53-day period. For the first 29 days, the leakage varied between 43 and 56 gpd per in. of diameter per mile. A leaky air valve was repaired and the leakage dropped to between 28 and 33 gpd per in. of diameter per mile for the remainder of the time (or about 37 per cent of the limit). It is evident that by the time the tests were made, the joints had already "taken up."

The same year, a line consisting of approximately 7,400 ft of 16-in. and 300 ft of 12-in. pipe was laid. The allowable leakage was only 50 gpd per in. of diameter per mile at a pressure of 115 psi. When the first test was made on July 7, 1936, the leakage was 388 gpd per in. of diameter per mile. It dropped rapidly until, on July 15, it was only 142 gpd per in. of diameter per mile. Tests were then discontinued until the middle of September, when readings showed the leakage to be only 32 gpd per in. of diameter per mile, well within the allowable 50 gpd.

In 1939 approximately 7,100 ft of 20-in. and 500 ft of 24-in. pipe were laid and tested at 90 psi with an allowable leakage of 100 gpd per in. of diameter per mile. The first test was made May 12, 1939, showing a leakage of 225 gpd per in. of diameter per mile. This amount decreased until, at the time of the last test on June 6, it was only 50 gpd per in. of diameter per mile, or half of the limit.

Thus, it appears that joints made with sulfur-base compounds usually have very high initial leakage that decreases in a parabolic curve, and practically levels off in from two to six months.

An unusual failure of some new joints is worth recording. A few days after the mains were chlorinated and placed in service, leaks occurred all along the line. When the joints were dug up, it was found that many of them had pushed out. A pressure-recording gage was placed on the line, and pressures double the line pressure were recorded. Investigation finally disclosed that the automatic closing of cone valves at a pumping station was causing water hammer. Hand operation of the cone valves was started, the water hammer ceased, and no further leaks developed. These valves are still operated manually under normal conditions, but in an emergency their operation is automatic.

Mechanical Joints

In June 1950, after discussions with other cities—notably Richmond, Va.—the use of mechanical-joint pipe was begun. The pipe is built to the specifications of the Cast Iron Pipe Research Assn., as revised in 1950. The gaskets are of rubber, either natural or synthetic, with a duck back and lead tips. The cast-iron bolts are heat treated and have tee-heads with hexagonal nuts. Both bolts and nuts are coated with a rustproofing compound after threading or tapping. The pipe

was first installed by one of the commission's pipe gangs. On the first few lines every joint was left uncovered and inspected for leakage after the line was charged. All joints were found to be completely dry. The pipe is now used extensively on contract work and is favored by both the commission and contractors.

On all water main projects except large supply lines, the commission furnishes the pipe. With cast-iron bell-and-spigot pipe, the contractor furnishes the jointing material, whereas with mechanical-joint pipe the joint accessories are bought with the pipe. The cost of the pipe is therefore 10 to 12 per cent greater than that of bell-and-spigot pipe, but this differential is partially offset by the fact that the bid prices for laying mechanical-joint pipe are almost 10 per cent lower. Against the slight additional expense of the mechanical-joint pipe may be weighed the advantages of very low leakage, more sanitary joints, and more flexible joints, which decrease the possibility of breaks caused by uneven soil settlement. The mechanical-joint pipe was proved more sanitary in the summer of 1951, during which there were less rechlorinations than in any previous summer despite a large volume of work. During this period, the majority of the pipe laid was of cast iron with mechanical joints. In addition, as there are no joints to pour but only bolts to tighten, mechanical-joint pipe is much easier to handle in wet ditches than is bell-and-spigot.

The following procedure is required in making mechanical joints:

1. Wet and clean the inside of the pipe ends with clean, fresh soapy water.
2. Slip the gland on the spigot end of the pipe with the lip extension of the gland toward the joint. Slip the gasket on the pipe with its thick edge toward the gland.
3. Wet the gasket thoroughly with soapy water and push the spigot end to its seat in the bell.
4. Press the gasket into place within the bell, being sure that it is evenly seated around the joint.
5. Move the gland into position for bolting, insert the bolts, and tighten up the nuts finger tight, making sure that the gland does not ride the pipe at any point.
6. Using a torque wrench set between 50 and 60 lb, tighten the bolts 180 degrees apart in order to bring up the gland evenly all around.

Pipe Coating and Lining

Two contracts under which approximately 18,200 ft of 36-in. diameter and 34,200 ft of 42-in. diameter steel pipe were laid, chiefly across country, from the Patuxent Filtration Plant to the Wheaton standpipe have just been completed for the commission. The exterior of this pipe was given a $\frac{1}{16}$- to $\frac{1}{8}$-in. coating of coal-tar enamel before shipping. Afterwards, a coating of whitewash was applied. The wall thicknesses of the pipe varied between $\frac{5}{16}$ and $\frac{1}{2}$ in. When the pipe was distributed along the right of way, it was not allowed to rest on the ground, but was supported and wedged on wood blocks. Before the pipe was laid, samples of the soil were obtained at approximately 400-ft intervals along the line and tested for specific acidity and pH in the commission's laboratories. In a few samples, a pH value below 5 was found. Retests were made at closer intervals in these areas, and no badly corrosive soil was found. If corrosive soil had been found, sufficient

lime would have been placed in the backfill to raise the pH to a noncorrosive value.

Just before being lowered into the trench, the pipe was given a spark test that detects the smallest pinhole in the coating. A few rather large pieces of the coating cracked off the pipe while it was lying along the right of way, probably as a result of sudden changes in temperature, rough handling, and defects in the coating. The holes were patched with bituminous enamel. A special wide sling was used by the crane for lowering the pipe into the ditch. Dresser couplings were used to make the pipe joints.

After being placed on the pipe, the coupling, which had been primed at the factory, was given a bituminous-enamel coating. A semicircular pan was placed under and around the bottom of the pipe and the enamel poured over the coupling in such a manner as to cover the top half completely and submerge the bottom half of the coupling in the enamel caught in the pan. The excess enamel in the pan was then drained off and returned to the heating kettle.

After the pipe had been laid and backfilled, a complete inspection of the interior of the entire line was made. A few dents that were found in the pipe were hammered out, and the pipe was uncovered to check the condition of the coating, which was, however, undamaged. The dents are believed to have been caused by clods of frozen earth which were inadvertently left under the pipe.

The pipe was then tested for leakage. The allowable leakage was 100 gpd per in. of diameter per mile. The required test pressures varied between 120 and 140 psi at the gage, with a maximum of 200 psi in some of the deep valleys the line crossed. The first test required that the line meet the allowable leakage for 3 hr.

The specifications allowed, and one of the two contractors on the job used, sulfur jointing compounds on the cast-iron valves and fittings. The other contractor went to the additional original expense of using lead. The first leakage test was applied shortly after the line had been laid. The sulfur jointing compound leaked badly and many joints had to be cut out and repoured with lead before a satisfactory test could be obtained. The lead joints gave no trouble. It appears to be bad policy to put high pressure on newly laid lines on which sulfur jointing compounds have been used.

After this preliminary test was satisfactorily completed, the line was drained, cleaned with wire brushes and coarse burlap, and lined with a cement coating. A lining between $\frac{3}{16}$ and $\frac{1}{4}$ in. thick was applied by a centrifugal process. The entire line was inspected after lining, and some test sections were cut out to check the thickness and quality of the interior. As soon as practicable, and within 48 hr after the lining was placed, water was introduced to allow curing. A minimum of fourteen days was allowed for curing except on the last section of the line, which, because of a large seasonal water demand, was needed for service as soon as possible. On the assurance of the lining firm that the lining would be satisfactory, only three days were allowed for curing this portion of the pipe.

The final 24-hr leakage tests were then made, and both lines passed easily. On the line jointed with sulfur-base compound, the leakage was 62.4

per cent of the permissible limit; on the line jointed with lead, it was 35.4 per cent of that permissible.

The commission had laid a large amount of steel and cast-iron pipe with bituminous-enamel lining. In 1939, 8,300 ft of 30-in. steel pipe were laid with both bituminous lining and coating. The pipe was cleaned thoroughly and then washed with high-test gasoline to remove any grease or oil. A coal-tar-base bituminous primer was then brushed on. From 24 to 144 hr later the bituminous enamel lining was installed. It was applied by a centrifugal process, and a thickness of $\frac{3}{32} \pm \frac{1}{32}$ in. was required. The pipe was also spark-tested. The lining was satisfactory except for a few blisters that developed, mostly at the ends of the pipe sections. It was found that if bituminous-lined, cast-iron pipe were not used shortly after lining, the lining had a tendency to peel. Once in the ground the bituminous lining appears to be satisfactory; plugs from taps made on mains fifteen years old show the lining to be in good condition. Inasmuch as pipe must often lay in the commission's yard for some time before use, however, the policy of buying all cast-iron pipe 12 in. and larger in diameter cement lined has been adopted. On cast-iron pipe less than 12 in. in diameter, coal-tar-pitch varnish with oil added is specified, and is applied in accordance with Specification AWWA C204 (1).

Concrete Pipe

In 1943 the commission built a 30-in. water supply line 48,900-ft long of steel cylinder, prestressed concrete pipe with bell-and-spigot joints. Specific acidity and pH tests were made on the soil in the same maner as on the steel lines. The pipe manufacturer set up a plant near the work and poured the pipe for this line and also shipped additional pipe to several other jobs within a radius of approximately 200 miles. The line was tested in sections at pressures ranging from 150 to 200 psi. The allowable leakage was the same as on the steel lines, 100 gpd per in. of diameter per mile. After repairing minor defects, the average leakage of the line was 60 per cent of the permissible limit.

In 1951 a 20-in. tap was made in this line while it was under pressure. Briefly, the major steps in making this tap required: installing the split tapping sleeve on the pipe, chipping off the exterior concrete within the sleeve opening, cutting the exposed prestressing wires, installing the valve, and setting up the tapping machine and making the tap. It took a total of three days to complete these operations. A representative of both the pipe manufacturer and the tapping sleeve and valve manufacturer supervised the work. Making such a large tap into a concrete line under pressure is perfectly feasible, but, like all large taps, it is very expensive.

Asbestos-Cement Pipe

The commission first used asbestos-cement pipe in 1941, when approximately 1,000 ft of 8-in. pipe were laid on one job by one of the commission's gangs. This pipe had a Dresser type coupling. The line has been entirely satisfactory except for some initial trouble in tapping that can be attributed to inexperience.

In the fall and winter of 1950 the commission laid about 16,000 ft of 6-in. and 8-in. asbestos-cement pipe using a sleeve type joint with rubber gaskets. Considerable trouble from leaks was

experienced both at the joints and in the pipe, which tended to break circumferentially. The mains were installed in new subdivisions under severe winter conditions. At this time of year the backfilled trenches become, and remain, thoroughly water-saturated, and the unpaved street surfaces are, in general, a sea of mud. Shortly after being laid, the mains were subjected to unusually heavy loading as material trucks, large bulldozers, and pans sank into and sometimes became stuck in the muddy trenches. Combinations of these conditions plus inexperience in laying the asbestos-cement pipe probably caused most of the trouble.

The commission now installs asbestos-cement pipe only from the middle of May until the middle of November, and uses specifications that do not permit the pipe to be laid on wood blocks or earth pads. The pipe is laid directly on the trench bottom (or on compacted gravel refill if the trench bottom is very wet or otherwise unsatisfactory), and holes are dug at the joints to obtain proper clearance and facilitate the coupling installation. After the coupling is installed, a feeler gage is used to determine that the gaskets are in their proper position.

As mentioned previously, the first 2 ft of the ditch in which the asbestos-cement pipe is laid are required to be tamped in 4-in. layers. Another very important requirement, not specified on the previous installations and intended to eliminate some circumferential breaks, is that short lengths—between 2 ft 2 in. and 4 ft 4 in.—of asbestos-cement pipe be used as connections to cast-iron fittings. This measure provides flexibility if the heavier cast iron settles. It is believed that the current method of installation will prove satisfactory. Asbestos-cement pipe has important advantages of economy, light weight, simple joints, and high resistance to corrosion.

Acknowledgments

The author wishes to express his great debt to those in the Washington Suburban Sanitary Commission who have helped him in the preparation of this paper: Harry B. Shaw, Chief Engr.; H. Roland Devilbiss, Deputy Chief Engr.; Justin E. Farrell, Acting Construction Engr.; Leroy Sasscer, Sr. Asst. Engr. in charge of inspectors; and many others.

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1. Standard Specifications for Coal-Tar Enamel Protective Coatings for Steel Water Pipe of Sizes up to But not Including 30 Inches—AWWA C204. Am. Water Works Assn., New York (1951).

The author has provided a valuable record of the experiences of the Washington Suburban Sanitary Commission in the installation of various types of pressure pipe. It should be noted that the record covers the conditions met by one utility in one location. These conditions and results may differ from those of other utilities or in other locations.—ED.

Coagulation With Ferrous Sulfate

By J. S. Gettrust

A contribution to the Journal by J. S. Gettrust, Supt., Water Purification and Pumping, and C. O. Hostetter, Chemist, Akron Water Works, Kent, Ohio.

FERROUS sulfate is the end product of the reaction between iron and sulfuric acid. The salt is soluble in water and, when crystallized and dried, has a greenish color. In this commercial state it is commonly called copperas.

Before iron is fashioned into final manufactured products, the stock must be cleaned. It is common practice to clean the stock of scale and rust by dipping it in hot sulfuric acid. In time, the acid becomes exhausted and the resulting ferrous iron solution is discarded. This expended pickling liquor is the basis of commercial ferrous sulfate.

There are two methods by which ferrous sulfate can be used as a water coagulant. The first is the old lime and iron process that has been in use many years but is gradually losing ground to other coagulants, mostly because of its inability to coagulate colored waters. In this method the ferrous sulfate is changed to ferrous hydroxide by the addition of lime and then the ferrous hydroxide is oxidized to ferric hydroxide by the natural oxygen in the water.

The second method utilizes chlorine to oxidize ferrous sulfate to ferric sulfate. Ferric chloride is formed in the reaction. The resulting ferric compounds, called chlorinated copperas, are then added to the water to be coagulated. These iron compounds react with the natural bicarbonates to form the hydrates. This reaction is similar to that of aluminum sulfate.

The author's plant uses the raw expended liquor from a steel mill. A contractor transports the acid solution in a wooden tank truck to the filter plant where a wooden tank has been provided for storage. The pickling liquor is measured by a flowmeter and is injected into the chlorinator discharge hose through a hard rubber injector. The oxidizing reaction takes place in a 50-ft coil of 3-in. rubber hose before the solution is applied to the raw water. Theoretically, 1 lb of chlorine will oxidize 7.8 lb of ferrous sulfate. In practice, however, 20 lb of chlorine per mgd is fed with every 17 ppm of iron dose, or approximately 1 lb per 7.2 lb of iron.

The amount of ferrous sulfate available does not permit its exclusive use. The average iron dose is about 9 ppm. The remainder of the required coagulant dose is made up with alum. The two coagulants work well together—in fact there are times when the combination of the two works better than either one alone.

Plant results are practically the same for each coagulant but the cost for iron is considerably less than that for alum. At 1950 prices, a ton of alum costs $35.90 and a ton of chlorinated cop-

peras, \$18.20 (\$9.75 for iron and \$8.45 for chlorine). Unit costs were \$0.08 per ppm of chlorinated copperas and \$0.16 for alum. The savings in 1950 were \$10,465 and the total saving from inception of this method in 1935 was estimated at \$120,000.

The pickling liquor as received contains approximately 1 to 2 per cent free sulfuric acid and has an average concentration of 3.5 lb of ferrous sulfate per gal. Extra quantities of lime must be added after coagulation to compensate for the pH reduction. The author feels, however, that the initial pH reduction aids coagulation so that smaller doses can be utilized to coagulate the water properly. The cost of the added lime dose is therefore offset by decreased alum doses.

Discussion

Wendell R. LaDue

Supt. & Chief Engr., Bureau of Water and Sewerage, Akron, Ohio.

Although both chlorine and filter alum are in short supply, hundreds of tons of ferrous iron solution are being wasted. Thus, the use of ferrous sulfate has a definite place in treating water supplies as well as in eliminating a byproduct waste disposal problem. A recent news publication emphasized the difficulty in disposing of ferrous sulfate solutions. The pickling liquor from a California tin plate mill is dumped a minimum of 20 mi at sea. A craft with a total capacity of 197,000 gal must make two or three trips a week to dispose of this byproduct.

This single incident illustrates not only a waste disposal problem, but a costly transportation problem and an economic waste in the destruction of valuable material. It certainly should be brought to the attention of water and sewerage operators.

The experience of the writer's plant with ferrous sulfate solution has been successful and profitable. Although the iron and lime process is not beneficial in treating raw water which has a high color, chlorinated copperas is decidedly helpful. The iron and lime process is useful, however, if turbidity and not color is the problem. With colored water in the normal pH ranges, the intermingling of the two processes—ferrous sulfate as copperas and alum coagulation—is practical and beneficial—in fact, as the author states, the two sometimes work better together than either one alone.

The writer feels that this information should materially benefit the water works industry, which is now suffering from a shortage of alum. It seems likely that all large industrial cities would have access to pickling liquor.

Observations on the Operation of a High-Rate Clarifier for Turbidity Removal

By H. K. Gidley

A paper presented on Oct. 5, 1951, at the West Virginia Section Meeting, Charleston, W.Va., by H. K. Gidley, Director, Div. San. Eng., West Virginia Dept. of Health, Charleston, W.Va.

IN September 1950, Moorefield, W.Va., a small town having a population of 1,401, placed in operation a new 400-gpm filter plant using water pumped from the South Fork of the South Branch of the Potomac River. The Moorefield plant (Fig. 1) is of conventional small-plant design.

A year later in September, 1951, Petersburg, W.Va., population 1,874, also placed in operation a new 400-gpm filter plant using water pumped from the South Branch of the Potomac River. The Petersburg plant (Fig. 2) deviates from the usual practice in using a high-rate upflow "sludge contact" type of clarifier. The basic design factors of both the Moorefield and Petersburg plants are given in Table 1.

The Moorefield and Petersburg plants, although only 12 miles apart, do not treat water from the same stream, but are on major tributaries of the same stream, the waters of which have similar physical and chemical characteristics. Table 2 gives the chemical characteristics of the waters treated in the two plants. Neither softening nor any other special treatment is employed in either plant; coagulation is used for turbidity removal only. The South Branch of the Potomac carries a greater turbidity load than the South Fork, but both streams are clear most of the time. Neither plant is equipped for the accurate measurement of low turbidities.

Engineers from the West Virginia Dept. of Health have observed and compared the performances of the two plants, with particular attention to the operation of the high-rate clarifier for turbidity removal. The observations presented refer to the treatment of a certain type of water and to small "one-man" water systems. The sources of water are typical of the mountain streams of West Virginia. These streams are clear most of the time, but are flashy, with moderately high turbidities occurring and subsiding rather quickly.

The two communities are similar in many respects and have had the same water supply problems. Both have had a slow but steady growth during the past 20 years and have outgrown their initial sources of supply, which were large-diameter, shallow-dug infiltration wells. These wells were located in the river flood plain of the South Branch of the Potomac and penetrated thin strata of water-bearing gravel. When the wells became inadequate, both communities were fortunate in having streams with an abundant natural flow available for development.

TABLE 1

Basic Design Features of Petersburg and Moorefield Plants

	Petersburg	Moorefield
Chemical Feeding	Alum—dry feed Lime—dry feed Activated silica—solution feed Chlorine—solution feed	Alum—dry feed Lime—dry feed Chlorine—solution feed
Mixing	Mechanical mix and sludge contact in clarifier	Fixed baffles Over and under retention—5 min Velocity—0.6 fps
Settling	Clarification in high-rate upflow unit Theoretical retention of clarifier—52 min Surface loading—2,400 gpd per sq ft	Horizontal rectangular basin with central longitudinal around-the-end training wall Theoretical retention—250 min Surface loading—430 gpd per sq ft
Filtration	Two rapid sand filters Filter surface of each unit—100.75 sq ft	Two rapid sand filters Filter surface of each unit—99.65 sq ft

High-Rate Clarifier Plans

When the Petersburg plant plans were presented to the West Virginia Dept. of Health for review, the department pointed out to the consulting engineer and the municipal officials that a high-rate clarifier would present operating problems with the water to be used, and predicted that the cost of operation would be higher than that for a plant using a settling basin of conventional design. The department had had successful experience with high-rate clarifiers in iron removal and softening operations, but the heavy slurry or sludge present with such treatment would not normally be present during the low turbidity intervals which would predominate on the South Branch. The department felt that, although the slurry concentration necessary for high-rate clarification could be established, it would require heavier chemical treatment and a degree of control which would tax the ability of the typical small-plant operator.

The natural site for the proposed Petersburg plant was on city property adjacent to the old infiltration well. This site presented a foundation problem as a relatively deep excavation through an alluvial deposit was required in order to reach firm bearing. The consulting engineer recommended a compact vertical design employing a high-rate clarifier to reduce construction costs. After considering the site conditions and a statement from the municipality that it was willing to accept higher operation costs in exchange for an anticipated lower installation cost, the department issued a certificate of approval.

Cost Comparison

It is impossible to compare closely the costs of the conventional Moore-

Fig. 1. Conventional Design Plant at Moorefield

The plant is located on the South Fork of the South Branch of the Potomac River

field plant with the high-rate Petersburg plant. The Moorefield plant contract price was approximately $55,000, whereas the Petersburg contract price was approximately $59,000, but the apparent advantage of the conventional design can be attributed to site differences and certain variations in design. In small high-rate plants, any savings resulting from the smaller structure are offset to a substantial degree by the cost of special equipment.

High-Rate Plant Operation

When the high-rate plant was placed in operation in September 1950, a technical representative of the equipment manufacturer was present for a few days to adjust the equipment, establish an operating procedure, and instruct the local operator. The initial chemical dosages required to establish and maintain the slurry in the clarifier were high. Operation was further complicated by the intermittent pumping cycle necessitated by the limited elevated storage of the distribution system. This off-and-on cycle made it difficult to achieve uniform operation. The plant was operated as intended by the designer for a period of three months. It consistently produced a satisfactory effluent with acceptable filter runs during this period, but the amount of coagulant required to maintain a slurry of proper density was high. Table 3 compares the chemicals used in the conventional and high-rate plants.

TABLE 2

Chemical Characteristics of Waters Treated in Petersburg and Moorefield Plants

	Petersburg South Branch Potomac River	Moorefield South Fork of South Branch Potomac River
Alkalinity—*ppm**	85	73
Hardness—*ppm**	108	78
pH*	7.3	7.3

* Averages for 4 weeks in August 1951.

In November 1950, plant operation at Petersburg was interrupted by an intake failure which could not be immediately corrected because of high water. As an emergency measure the town returned to the old well as a source of supply, but treated the well water in the new plant. Inasmuch as the well water was clear at all times, no effort was made to maintain a sludge blanket in the clarifier. A minimum dose of alum was used to establish a

Fig. 2. High-Rate Plant at Petersburg

The plant is located on the South Branch of the Potomac River.

light floc which was allowed to carry over to the filter. A large savings in chemicals was effected by this procedure, and led to the establishment of the following treatment program which is now used.

1. When sufficient water is available in the old well, it is used as a source of supply with a coagulant dose of 100 lb of alum per mil gal.

2. When the well supply fails the river intake is used. After checking for a year, a minimum alum dose of 300 lb per mil gal was established as the most economical dose that could be used; with this dosage some floc is carried over to the filter.

TABLE 3

Alum Used for Coagulation in Petersburg and Moorefield Plants

	Water Treated *mil gal*	Alum *lb*	Max. and Min. Dosages *lb per mil gal*	Average Dosages *lb per mil gal*
Petersburg	5.14	1,807	288–390	352
Moorefield	6.09	497	53–108	82

The high-rate plant is equipped with a solution feeder for the dosing of activated silica. Jar tests by the West Virginia Dept. of Health engineers indicated that silica could sometimes be used to improve floc density. Trial runs on a plant scale using silica, however, did not improve plant performance sufficiently to justify its use.

Filter runs should offer a basis of comparison of the efficiency of the conventional and high-rate basins, but they are probably not a sound index for the Moorefield and Petersburg plants inasmuch as neither plant controlled backwash strictly by loss of head. In small plants operating a few hours each day, it is common practice to wash filters at least once each week, regardless of loss of head. An examination of the reports submitted to the West Virginia Dept. of Health indicates filter runs varying from 30 to 60 hours for both plants.

Conventional Design Advantages

The conventional settling basin seems to have several advantages for the small plant. One of the best features of prolonged settling is the ability of the settling unit to function fairly well at a 100 per cent overload. This feature has been a valuable asset in many small plants in West Virginia which have had to operate at substantial overloads pending the initiation of new construction.

A large settling basin is advantageous if intermittent or part-time operation is practiced. After a plant has been inactive for several hours the basin contains clear, settled water which has benefited from prolonged quiescent settling. The buffering action of this basin of clear water lessens the urgency of optimum coagulation of the incoming raw water.

Features of the high-rate clarifier which are desirable but generally lacking in the small conventional basin are efficient mechanical mixing and well designed effluent weirs. Automatic sludge drawoff may be an advantage with some waters, but the volume of sludge produced in treating a typical West Virginia stream is small; the standard practice of semiannual cleaning seems to be satisfactory.

Conclusions

1. The high-rate clarifier is more difficult to operate than a conventional low-rate basin with gravity settling. This is an important consideration in

a small plant where the technical training and skill of the operator may be limited.

2. When treating the low-turbidity waters typical of many West Virginia streams, the amount of coagulant or special coagulant aids required to maintain a dense slurry in the clarifier will be greater than that required for effective clarification in a conventional basin.

3. A conventional low-rate settling basin is more stable in operation than a high-rate clarifier and will accept a substantial overload, important characteristics in small-plant design.

Correction

In the "Tentative Standard Specifications for Bauxite" which appeared in the July 1950 JOURNAL, the line italicized below was omitted. Section 9C.2.4 on p. 714 should be amended to read:

Add 20 ml of preventive solution (9C.1(b), dilute to 150–200 ml, and titrate. . . .

Effect of Lime-Treated Water Upon Survival of Bacteria

By M. L. Riehl, H. H. Weiser, and B. T. Rheins

A contribution to the Journal by M. L. Riehl, Chief Chemist, Ohio Dept. of Health Laboratories, and H. H. Weiser and B. T. Rheins, both of Dept. of Bacteriology, Ohio State University, Columbus, Ohio.

THE destruction of many waterborne bacteria in water previously treated with excess lime was observed by Houston (1) more than 40 years ago. Later, Hoover (2) made further studies and also confirmed and extended the work of Houston. Other research has been conducted and reported by independent investigators interested in water treatment in Ohio. Streeter (3) has been interested in this aspect of water treatment and has reported his findings in various scientific journals.

It was not until World War II, when the question of protecting our water supplies from sabotage was of great concern, however, that a comprehensive study was made under Streeter's supervision. The bactericidal efficiency of excess lime in water treatment was challenged by many public health officials. Wattie and Chambers (4) studied this problem and found that the high pH values of the treated water exerted a marked bactericidal effect upon certain test organisms. In order to augment the studies of Wattie and Chambers, a research project which entailed the investigation of additional variables was outlined. This investigation has been in progress for the past two years (1949–51) in cooperation with the Dept. of Bacteriology, Ohio State Health Laboratories, and the Ohio State University Research Foundation.

The purpose of the authors' investigation was to determine the survival of selected bacteria in lime-treated water samples from different sources. The following variables were studied: [1] the survival of selected bacteria when added to various water samples; [2] the effect of pH on the survival of the organisms; [3] the influence of the composition of the test waters—distilled, hard, soft, and turbid; and [4] the influence of temperatures—2 to 5 C, 15 C, and 25 C.

Experimental Procedure

A laboratory strain of *Esch. coli* was added to distilled water samples in quantities of 1,000 organisms per ml. Varying amounts of calcium hydroxide were added to each sample, except the control, to obtain the desired pH range of 7.3 to 11.7. The samples were mixed rapidly for 5 min and then slowly for 2 hr. A portion of this mixed sample was removed for chemical analysis. One series of water samples was incubated at 5 C, another at 15 C, and a third set at 25 C. Bacterial counts were made 1 min after

the test organisms were added, then at 30-min intervals for 2 hr, and then hourly for 10 hr. Counts were made on a few samples for up to 100 hr. All agar plates were prepared in duplicate, incubated for 24 hr at 35 C, and then counted.

Results

Figure 1 shows the percentage of survival of the test organism at different levels of pH over an 8-hr period. Apparently at the higher pH ranges of 11.7 at 5 C, 11.4 at 15 C, and 11.2, 10.8, and 10.3 at 25 C, the test organism did not survive more than 30 min regardless of the temperature. At the lower pH range, *Esch. coli* showed marked diminutions, however, as the 8-hr period ended. A temperature of 5 C showed the greatest percentage of survival, whereas 15 C was more effective in destroying the organisms but only at the high pH levels; at 25 C only the test organism in water at pH 7.5 was able to survive up to 8 hr, and then only in small numbers.

The survival of a freshly isolated strain of *Esch. coli* in different kinds of water ranging from distilled water, soft, turbid, and hard water, the latter with a hardness of 800 ppm, was studied at 15 C. The results are shown in Fig. 2. The destructive effect on the test organism of a high pH was again observed. It appears that soft water at different pH levels is effective in destroying *Esch. coli* within an 8-hr period. In the other types of water samples, the organisms survived throughout the 10-hr period, although the percentage of survival varied with each type water and pH level.

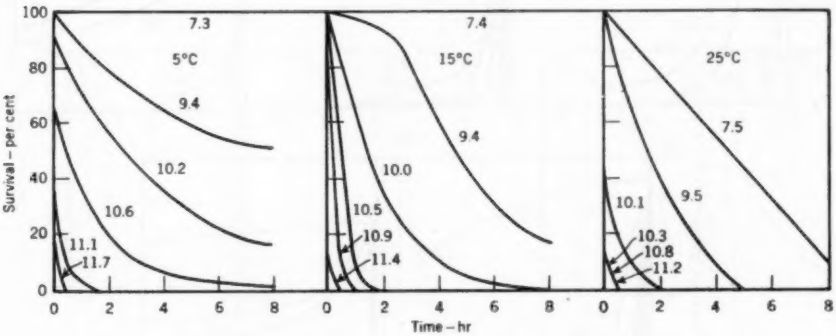

Fig. 1. Survival of *Esch. coli* at Different pH Levels

The organisms were incubated in distilled water at 5, 15, and 25 C.

The results of a comparison of the survival of *Salmonella sp.* (Type Montivideo) and *Salmonella typhosa* in different kinds of waters at 2 C and 12 C are shown in Fig. 3. In distilled water, *S.* Montivideo remained viable in appreciable numbers at the lower pH levels throughout the 10-hr period; at pH 11.0 the organisms were destroyed in approximately 2 hr at 2 C and 30 min at 12 C. At 12 C, however, when pH levels were maintained at 10.7, 10.3,

and 10.0, the organisms survived 4, 7, and 10 hr, respectively. At 2C and pH 11 the survival time was more than 2 hr. When pH levels of the water samples were held at 7.5, 10.1, and 10.6, the organisms survived up to 10 hr with no appreciable reduction in numbers. A pH of 11.1 required the the time was 3 hr, and at pH 10.8, more than 8 hr. When Scioto turbid water was compared with other types of water, a similar comparison was made. At pH 11.5 at 2 C, however, the time was 6 hr, whereas at pH levels 11.0, 10.5, and 10.0, a 10-hr period was required, but the number of

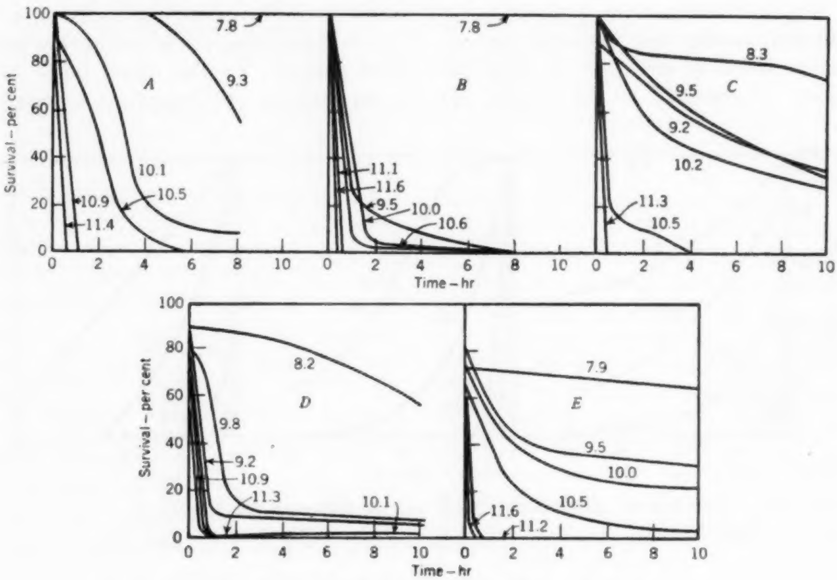

Fig. 2. Survival of *Esch. coli* in Different Media

The organisms were freshly isolated and incubated at 15 C. The italic letters refer to the following media: A—distilled water; B—Lancaster, Ohio, soft water; C—Scioto turbid water; D—Scioto clear water; and E—New Rome, Ohio, hard water.

10-hr period, but the number of organisms surviving was markedly less.

In Scioto River clear water at 2 C, the test organism survived the 10-hr observation period with no appreciable reduction of *S.* Montivideo at pH 8.2; at pH 10.4, 10.7, and 11.0 the survival time was greatly extended. At pH 11.4 the organisms survived 5 hr. The same general trend was noted when survivals at 2 and 12 C were compared, with the exceptions that at pH 11.3 organisms that survived was greatly reduced. At pH 8.5 at both 2 and 12C, there was no marked reduction of organisms up to the 10-hr period. At 12 C and pH 10.6, the organisms survived up to 10 hr, but in small numbers, whereas with the same temperature at pH 10.9, the time was 6 hr and at pH 11.1, 3 hr.

When *S. typhosa* was added to distilled and clear waters the results were not comparable with those attained with

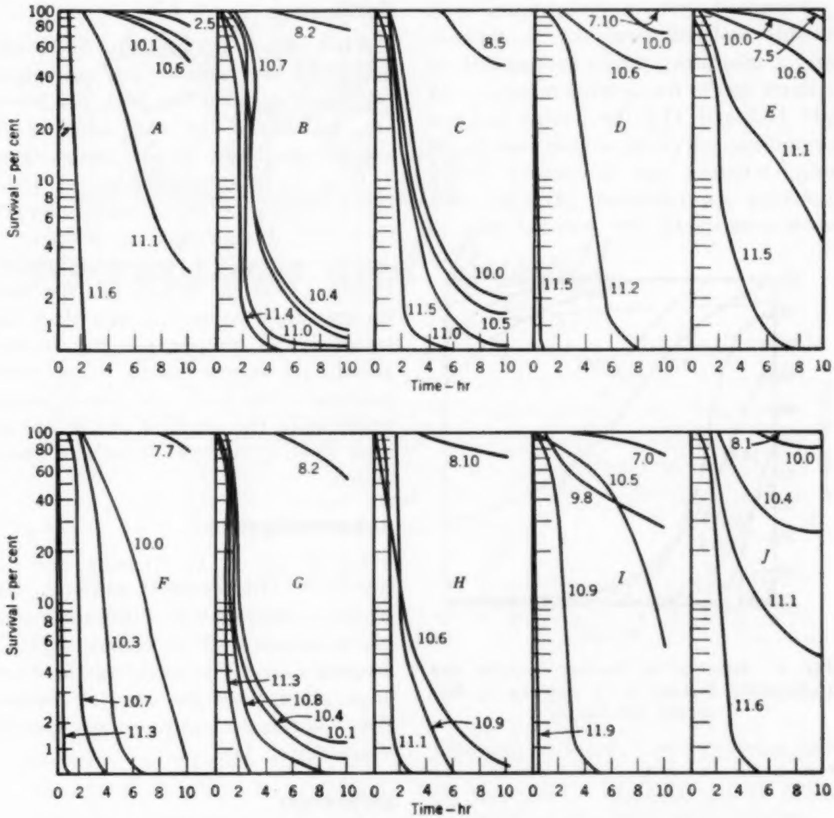

Fig. 3. Survival of *S.* Montivideo and *S. typhosa* in Different Media

The organisms were incubated at 2 and 12 C. Italic letters A, B, and C refer to S. *Montivideo at 2 C in distilled Scioto clear, and Scioto turbid water, respectively; letters D and F refer to* S. typhosa *at 2 C in distilled and Scioto clear water, respectively; letters F, G, and H refer to* S. *Montivideo at 12 C in distilled, Scioto clear, and Scioto turbid water, respectively; and letters I and J refer to* S. typhosa *at 12 C in distilled and Scioto clear water, respectively.*

S. Montivideo. At 12 C, *S. typhosa* survived 5 hr in clear water with a pH of 11.6. The same organism at 2 C and pH 11.5 survived approximately 8 hr. Otherwise at different pH levels the test organisms survived the 10-hr period although there was no correlation between the number of organisms surviving. In distilled water at pH 11.5, *S. typhosa* survived up to 1 hr at 2 C, and only 30 min at 12 C. At 2 C and pH 11.2, the organisms survived up to 8 hr as compared with 5 hr at 12 C and pH 10.0. Although the test organisms were able to survive up to the 10-hr period at the lower pH levels, the numbers varied markedly with each pH level.

Figure 4 gives a comparison of a freshly isolated strain of *S. typhosa* and a laboratory strain propagated on culture media for several months. At pH 11.7 and 11.5 the freshly isolated organisms survived approximately 60 min, whereas the laboratory strain survived approximately 30 min. Almost consistently the survival time of

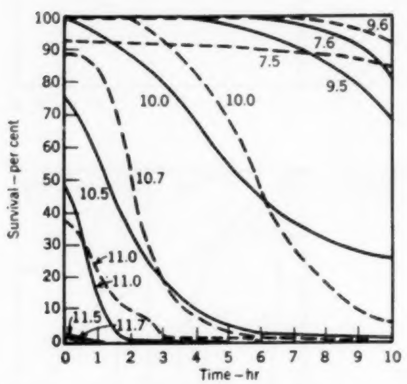

Fig. 4. Survival of Freshly Isolated and Laboratory Strains of *S. typhosa* at Different pH Levels

The organisms were tested in Lancaster soft water at 15 C. The broken line refers to the laboratory strain and the solid line to the freshly isolated strain.

the freshly isolated test organism was longer than that of the laboratory culture.

These results are significant from the public health standpoint in that recent pollution of a water supply would in all probability implicate an organism displaying resistance to chemical treatment similar to that of the freshly isolated strains.

Conclusions

Esch. coli, *S. typhosa*, and *S.* Montivideo do not survive for prolonged periods in water when high pH levels are maintained by the addition of excess lime. At a pH range from 11.0 to 11.5, a temperature of 15 C, and a holding period of slightly longer than 4 hr, this method was effective in destroying many of the test organisms. Freshly isolated strains of the test bacteria were more resistant than the same species propagated for several months on culture media. The composition of the water did not influence appreciably the survival of the organisms when a high pH level was maintained.

Acknowledgment

Leo F. Ey, F. W. Waring, and the late C. P. Hoover have served as the advisory committee in cooperation with the technical staff in conducting this investigation. The work was financed by a grant from the Div. of Sanitation, National Institutes of Health, Washington, D.C.

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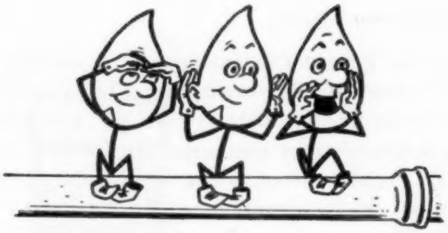

Percolation and Runoff

Kansas City, at this moment, stands in imminent danger of two floods—the first a disastrous one by a record-high Missouri River; the second, a beneficent one by a record-high AWWA registration. If we were certain that KC were as well prepared for the first record as we know it to be for the second, our crystal ball wouldn't look quite so much like the muddy Mo. Mo or no, though, we can report that these new officers and directors took office at the Board of Directors meeting on May 9:

President—Charles H. Capen, chief engineer since 1942 of the North Jersey Dist. Water Supply Commission, Wanaque, N.J. Born in Jersey City, N.J., in 1895, he received his civil engineering degree from Cornell Univ. in 1917.

His engineering career began upon his graduation with his appointment as resident engineer for N. N. Chester Engineers of Pittsburgh, Pa. In 1918 he designed water and sewer installations for the Navy while associated with Nicholas S. Hill of New York, and the next year joined the New Jersey Dept. of Health as assistant sanitary engineer. This connection was terminated in 1925 when he joined the North Jersey organization as assistant engineer, advancing to his present post with an interruption for military service during World War II, when he was principal sanitary engineer for the Second Corps area. He has also been engaged as a consultant on more than 50 water and sewerage projects.

Widely known in technical circles, he has been an AWWA member since 1930, and his Association activities are extensive. He was chairman of the AWWA Publication Committee from 1947 to 1951, Chairman of the Goodell Prize Committee and of the Fuller Award Society, and is or has been a member of the Convention Management Committee, the Water Rates Committee, the Sluice Gates Committee, and the Task Group on Industrial Water Use. In New Jersey, he was chairman of the section legislative committee that sponsored and helped obtain licensing and tenure legislation in the state, was the section's Fuller Award winner in 1938, its chairman in 1941, and its national director from 1947 to 1950.

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He served as chairman of the governor's Engineering Committee in 1938, reporting on New Jersey's water supply, was in charge of an interconnection survey for the state's major water supply systems in 1939, and in the same year was chairman of the advisory committee on licensing of water and sewage plant operators. He also served as chairman of the Joint Operation Board for emergency water supply operation.

His activities in many technical organizations other than AWWA and his many achievements have earned him such recognition as the Fuertes Medal, awarded by Cornell Univ., and the James F. Lincoln Award of the Arc Welding Foundation.

Vice-President—Morrison B. Cunningham, superintendent and engineer, Water Dept., Oklahoma City, Okla. Born in Greenville, Tex., in 1896, Cunningham served in the Navy during World War I and joined the Oklahoma City department in 1919, first as superintendent of construction, then, the following year, as distribution system superintendent. In 1922 he was appointed assistant superintendent and engineer, and after ten years in this position became assistant and then acting city manager. He was appointed to his present post in 1937.

An AWWA member since 1930, he became a member of the old Southwest Water Works Assn. in 1921. He became vice-chairman of its successor, the Southwest Section of AWWA, in 1939 and served as its chairman the following year. In 1948 he became the section's national director for a three-year term, and received its Fuller Award in 1949. He has also been an officer of AWWA's Water Works Management Div., chairman of the Committee on Radio and Mobile Communication Facilities for Water Works, and a member of the Committee on Water Works Administration and the Task Group on Erosion Control of the Committee on Watershed Protection and Maintenance. He is AWWA representative on the National Committee for Utilities Radio. Other technical societies in which he is active include the Oklahoma Society of Professional Engineers and the Engineering Club of Oklahoma, both of which he served as president and member of the Board of Directors; the Engineers Joint Council; and the Federation of Sewage and Industrial Wastes Associations. He has also been a member of the Governor's Board on Stream Pollution, the Engineers' Advisory Board for State Planning and Resources, and the Board of Interstate Compacts.

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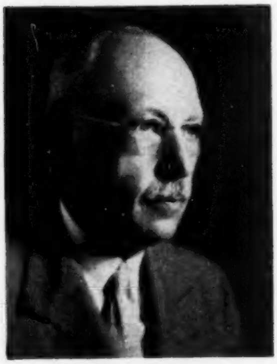

Treasurer—William W. Brush, editor of *Water Works Engineering.* Brush was born in Orange, N.J., in 1874 and was educated at New York Univ., from which he received B.S., C.E., and M.S. degrees. He served as engineer with the Brooklyn Water Dept., from 1894 to 1907, transferring to the New York Board of Water Supply in the latter year. In 1927 he was appointed chief engineer, and served in that capacity until 1934, when he retired after a cumulative total with the two organizations of 40 years of service. He then began his present affiliation with the Case-Shepperd-Mann Publishing Corp. and *Water Works Engineering* as editor of the latter publication.

His reelection as treasurer continues an unbroken record of 30 years of high office in the Association, for, except for a two-year interval in 1928–29 when he was successively vice-president and president, Brush

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has been treasurer since 1922. In that time also he has been active on many AWWA committees, and has served ex officio as a member of both the Board of Directors and its Executive Committee. He received the John M. Diven Medal in 1932 and in 1937 was made an Honorary Member.

Canadian Section—William Donald Hurst, city engineer and commissioner of buildings, Winnipeg, Man. Born in that city in 1908, he attended the University of Manitoba, from which he received the degree in civil engineering in 1930, and Virginia Polytechnic Inst., from which he received the C.E. degree in 1931.

While still an undergraduate, he joined the Hurst Engineering & Construction Co. as a junior engineer. In 1930, as a member of the Winnipeg Engineering Dept., he served as an inspector of reservoir construction. After a short interval for his graduate studies, during which he also acted as a teaching fellow, he returned to the Winnipeg Engineering Dept. and served in various capacities, becoming Engineer of Water

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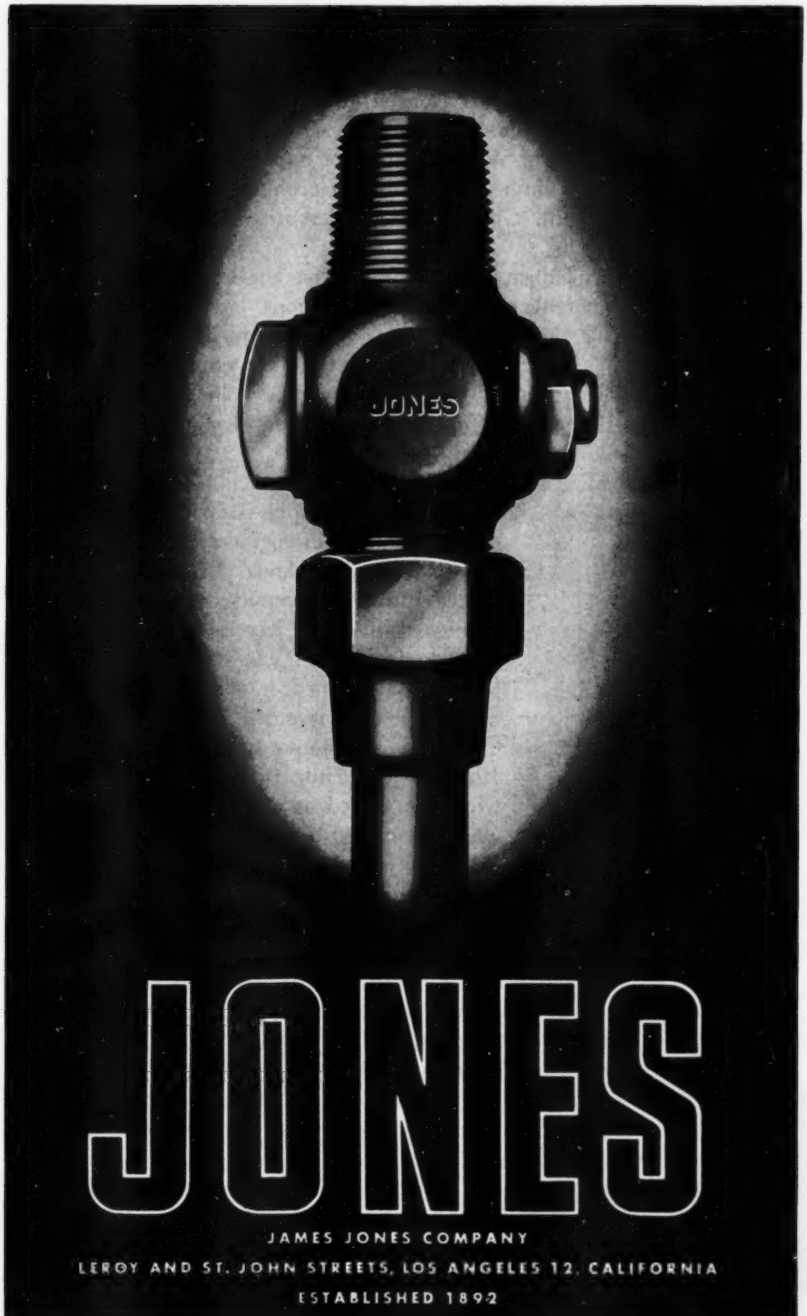
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Works in 1933 and assistant city engineer the following year. In 1935 he began service as secretary-engineer for the Board of Engineers of the Greater Winnipeg Water Dist., a connection he maintained until 1939. After an interval during World War II with the Royal Canadian Engineers, he was appointed city engineer of Winnipeg. Other appointments include that of commissioner of the Winnipeg-St. Boniface Harbour Commission, chairman of commissioners for the Greater Winnipeg Water Dist., and the Greater Winnipeg Sanitary Dist.

An AWWA member since 1934, he has been active in both the Canadian and the neighboring Minnesota Sections, having been trustee of the former and chairman of the latter. In addition, he has received the Minnesota Section's Fuller Award. Other technical groups in which he is active include the Engineering Institute of Canada, of which he was chairman of the Winnipeg Branch, the Canadian Institute on Sewage and Sanitation, and the Scientific Club of Winnipeg.

Indiana Section—Lewis S. Finch, vice president and chief engineer, Indianapolis Water Co. Born in Anna, Ill., in 1897, he attended Purdue University, from which he was graduated in 1921, receiving the civil engineering degree in 1931. He is a registered professional engineer in Indiana.

In the employ of the Milwaukee Sewerage Commission as senior engineer from 1923 to 1925, he joined the Indiana Board of Health, which he served as chief engineer until 1933, when he opened his office as a consultant. In 1942 he joined the Indianapolis Water Co. as principal assistant engineer, becoming chief engineer two years later. He was advanced to his present post in 1950.

In 1932, after a term as vice-chairman of the Indiana Section, he was elected its chairman, and in 1949 he received its Fuller Award. He has served on many AWWA committees, including the Committee on Water Works Administration, assigned to the Task Group Public Use of Watershed Areas, and the Committee on Water Main Extension Policy. In other areas he has served on the Ohio River Board of Engineers, the Great Lakes Board of Engineers, and the Advisory Board to Indiana Administrative Building Council. He is also serving on the Indiana Advisory Health Council and the Indiana Stream Pollution Control Board. He has been president of the Indiana Section of the American Society of Civil Engineers, the Central States Sewage Works Assn., and the Indiana Engineering Council.

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Iowa Section—Dale L. Maffitt, general manager, Water Works, Des Moines. A real native son, Maffitt was born in Des Moines in 1892, was graduated from Drake University there in 1914, and has been a member of the city's water works staff ever since, with a brief leave of absence in 1920–1921 to do graduate work in chemistry and chemical engineering at the Massachusetts Inst. of Technology. Having joined the Des Moines staff as assistant chemist, he was advanced to chief chemist in 1916, assistant general manager in 1926, and general manager—his present post—in 1933. He is a member of the Missouri River Ten States Committee, the Iowa Water Planning Committee, and has been president of the Engineers Club of Des Moines.

An AWWA Member since 1918, Maffitt now serves his second term as a member of the board, having been elected as his newly organized section's first director in 1946. He was also chairman, in 1938, of the predecessor Missouri Valley Section, and was vice-chairman and chairman of the old Finance and Accounting Div. of AWWA. The impressive list of his Association activities also include the chairmanship of the Fuller Award Society (for which he qualified by receiving his section's award in 1949), and of the committees on Public and Worker Relationships and on Pension and Retirement Plans. He is currently vice-chairman of the Water Resources Div. Other committees on which he has served include Water Works Practice, Federal Activities, Survival and Retirement Experience With Water Works Facilities, Social Security Legislation, Program, Fire Prevention and Protection, Valuation and Depreciation and Municipal Water Works Organization.

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Kansas Section—H. H. Kansteiner, manager of production and distribution, Waterworks Dept., Leavenworth, Kan. Born in St. Charles, Mo., in 1902, he studied mechanical engineering at the University of Missouri, receiving the B.S. degree in 1926.

He joined the Laclede Gas & Light Co. the next year as a cadet engineer, later becoming a sales engineer for commercial refrigeration, power plant and other supplies and services. In 1933 he joined the Public Works Board of his native St. Charles as superintendent of the Water Dept. He came to Kansas in his present capacity of production and distribution manager for the Leavenworth Waterworks Dept. in 1937.

An AWWA member since 1934, he was active in the old Missouri Valley Section and has served as vice-chairman and chairman of the Kansas Section. He was also chairman of a section committee studying the licensing or certification of water works operators.

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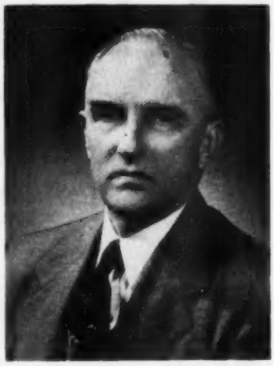

Missouri Section—Warren A. Kramer, chief, Water Supply, State Div. of Health, Jefferson City, Mo. Born in Franklin, La., in 1896, he attended the University of Mississippi, from which he was graduated in 1921; Louisiana State University, from which he received a master's degree in 1924; and Ohio State University, at which he obtained the doctorate in chemistry in 1927. He is a licensed professional engineer in Missouri.

While attending Ohio State, he did part-time research work under the direction of the late Charles P. Hoover at the Columbus water plant. In 1924 he joined the faculty of Louisiana State University, moving on to Ohio State the next year. In 1927 he joined the Dearborn Chemical Co. as chemical engineer and in 1930 became senior chemical engineer in charge of the St. Louis Chain of Rocks purification plant. He joined the State Div. of Health in 1938.

A member of AWWA since 1927, he has been Secretary of the Missouri Section continuously since its organization in 1946, and received its Fuller Award in 1950. He has been a member of the Committee on Capacity and Loadings of Water Treatment Processes.

North Carolina Section—George S. Rawlins, vice-president of the engineering and architectural firm of J. N. Pease & Co., Charlotte, N.C. Born (1904) and reared in Geneva, N.Y., he attended Cornell Univ., receiving the degree of civil engineer in 1925.

After a short period with Drexel Inst., Philadelphia, as an instructor in civil engineering, he became a design engineer for the Reading, Pa., Water Bureau. Two years later, in 1928, he joined the engineering and architectural firm of William H. Dechant & Sons, Reading, as an engineer. In 1932 he returned to the Water Bureau as assistant engineer, in which capacity he worked on the design of the Maiden Creek Filter Plant, later assuming charge of the operation of the supply. He joined the Chester Engineers of Pittsburgh in 1936 and came to Charlotte, N.C., in 1937 as engineer in charge of the city's water works design and construction. After maintaining his own consulting practice for two years, he closed his office to join the Pease

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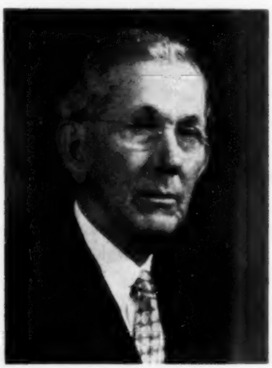
organization in 1941 in the capacity of partner, later becoming vice-president of the firm.

An AWWA member since 1939, he is affiliated with a number of other technical groups and has been a director of the Federation of Sewage and Industrial Wastes Associations, president of the Charlotte Engineers Club, and a director and also governor of the Professional Engineers of North Carolina.

Rocky Mountain Section—C. G. Caldwell, director of the Division of Sanitary Engineering and Sanitation, State Dept. of Public Health, Santa Fe, N.M. Born in Cliftondale, Mass., in 1912, he came to New Mexico for his schooling, receiving the civil engineering degree from the state university at Albuquerque in 1937. He also did graduate work in public health and sanitation at the University of California and then obtained a master's degree in sanitary engineering from the University of North Carolina in 1946. He is a registered professional engineer in New Mexico.

After a brief interval with the U.S. Forest Service and the New Mexico Highway Dept., he became a malaria control engineer upon leaving school in New Mexico. In 1937 he joined the New Mexico Construction Co. as a research and estimating engineer. The next year he joined the New Mexico Dept. of Public Health, with which he has remained ever since.

In the Rocky Mountain Section, he was successively section director, vice-chairman, and, in 1950, chairman.



Southeastern Section—Robert B. Simms, superintendent of the Water Works and Metropolitan Dist., Spartanburg, S.C. Simms served his water works apprenticeship for the Atlanta, Ga., supply in the capacity of meter reader, bookkeeper, cost accountant and assistant superintendent before coming to Spartanburg. He has been head of the latter utility since 1920.

A member of AWWA since 1922, he is now a Life Member. He has served on a number of Association committees, including the Forestation Task Group of the Watershed

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KENNEDY A.W.W.A.
WATER GATE VALVES
KENNEDY
AWWA
GATE VALVES
Conforming with Specifications
of the
American Water Works Ass'n

(Continued from page 16)

Protection and Maintenance Committee and the Weather Control Task Group of the Water Resources Development Committee. In 1951 he was awarded his section's Fuller Award. Other honors that have come his way include the presidency of the Southeastern Water & Light Assn. when application was made for AWWA section affiliation, and the renaming of the Spartanburg water plant in 1950 by the Commissioners of Public Works as the R. B. Simms Filtration Plant, in recognition of his outstanding vision and service.

Virginia Section—Xenophon Duke Murden, manager of the Portsmouth Water Dept., serving Portsmouth and Suffolk, Va. Born in Portsmouth in 1898, Murden has been serving that community ever since he left its public schools and, in 1917, became a meter reader for the Portsmouth, Berkeley, and Suffolk Water Co., which was later acquired by Portsmouth. Correspondence and extension courses with LaSalle Extension University and the University of Virginia supplemented his education, and he rose from post to post until, when he was made head of the department in 1941, he had had experience as bookkeeper, chief inspector, and assistant superintendent to round out his early meter reading career.

A member of AWWA since 1941, Murden has been chairman of the Virginia Section and is a member of the Engineer's Club of Hampton Roads.

West Virginia Section—Henry W. Speiden, head of the Dept. of Civil Engineering at West Virginia University. A native of Bluefield, W.Va., he was born there in 1903 and educated in its schools. He attended West Virginia University, from which he received both bachelor's and master's degrees in civil engineering (in 1925 and 1933, respecsively).

After completion of his undergraduate work, he was employed on the construction of a hydroelectric plant at Lake Lynn, Pa., and did highway work for Monongalia County, W. Va., and the state itself. In 1933 he joined the faculty of the university as an

(Continued on page 20)

(Continued from page 18)

instructor in the department of civil engineering, and he has retained that connection ever since.

An AWWA member since 1935, he was chairman of the West Virginia Section in 1943.

Wisconsin Section—Harold L. Londo, superintendent, Water Dept., Green Bay, Wis. Born in Wausaukee, Wis., in 1901, he obtained a civil engineering degree from the University of Notre Dame in 1924. He is a licensed professional engineer in the state of Wisconsin.

Londo's first post after leaving school was with the Pennsylvania Highway Dept., as an engineer. In 1927, after three years at this occupation, he became assistant city engineer for Green Bay, Wis., and remained in the service of that city through his transfer to the Water Dept. as assistant superintendent in 1935 and his promotion in 1941 to superintendent—the post he has held ever since.

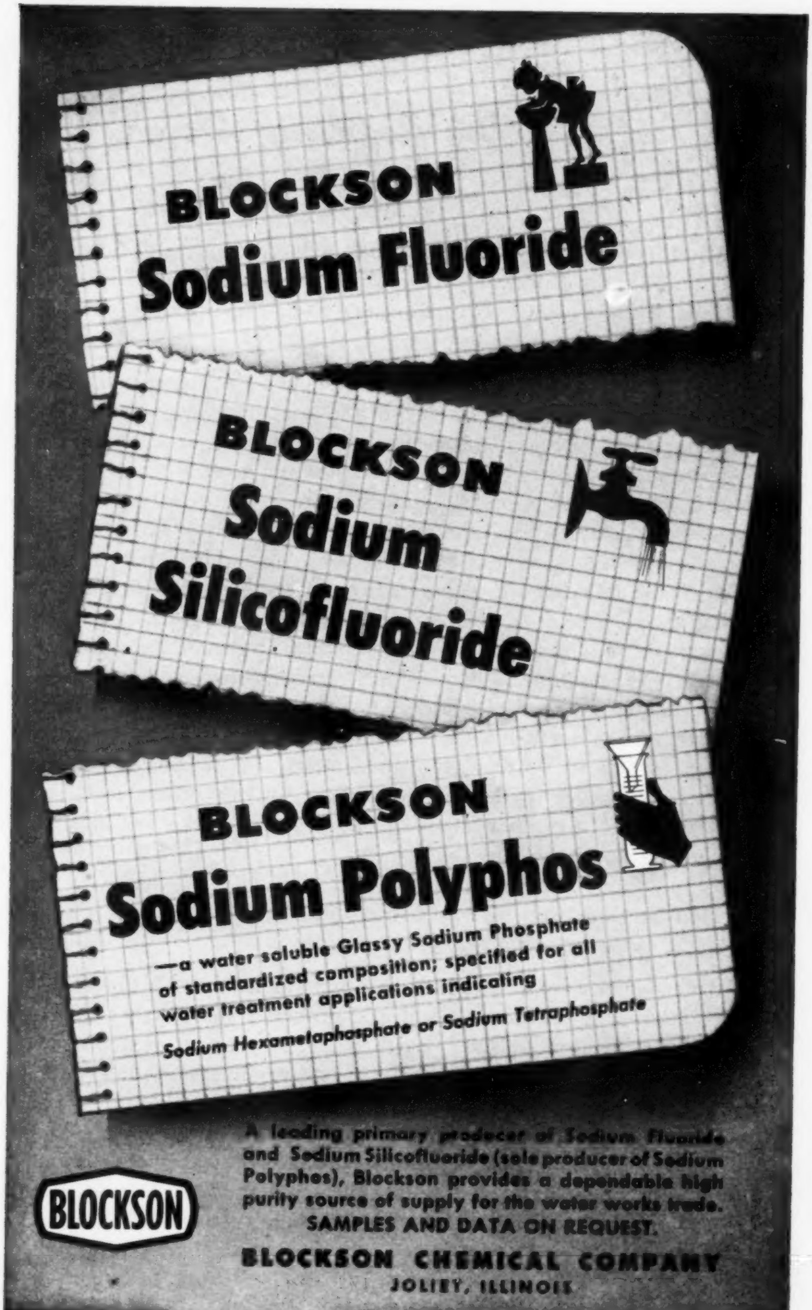
He has been chairman of the Wisconsin Section and in 1950 was recipient of its Fuller Award.

Manufacturer—Hubert Francis O'Brien, president and member of the Board of Directors of The A. P. Smith Manufacturing Co. of East Orange, N.J. Born in that community in 1910, O'Brien attended Princeton Univ., from which he received the Bachelor of Science degree in 1931.

In addition to his affiliation with the A. P. Smith organization, of which he has been president and director since 1941, he is a member of the board of the U.S. Pipe & Foundry Co. of Burlington, N. J. He has also been a director and member of the Executive Committee of the National Assn. of Manufacturers and a member of the Board of Governors of the Water & Sewage Works Manufacturers Assn., the organization he will now be representing on the AWWA board.

Joseph A. Snook has been appointed vice-president in charge of sales of Atlas Mineral Products Co., Mertztown, Pa.

(Continued on page 84)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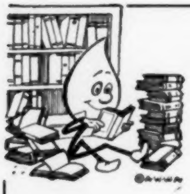


The Reading Meter

Civil Engineering Reference Book. *E. H. Probst & J. Comrie, ed. Butterworths Scientific Publications, London (1951) 1703 pp.; $22 from Butterworth & Co. (Canada) Ltd., 1367 Danforth Ave., Toronto 6, Ont.*

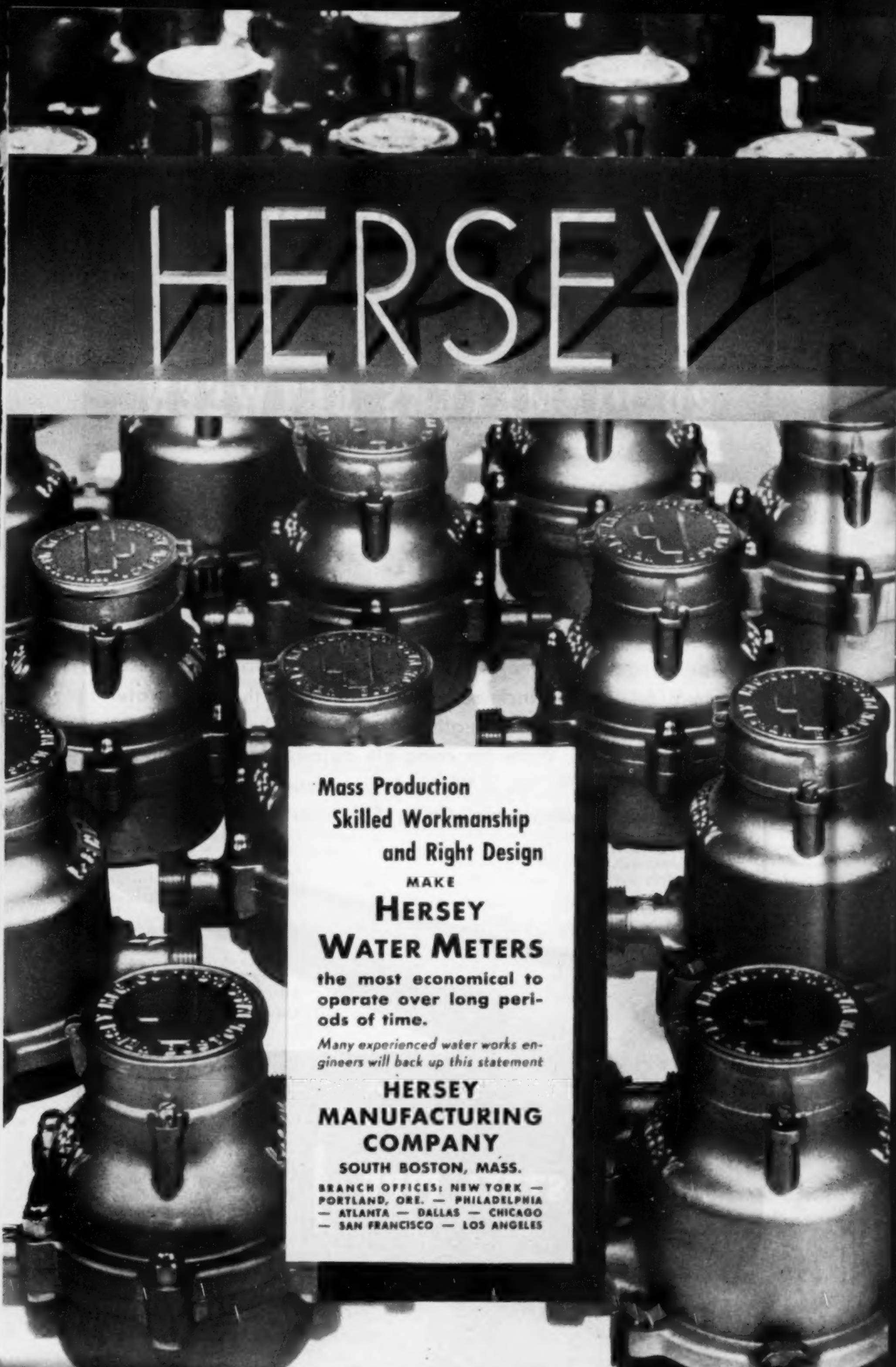
In addition to the chapter on water supply, this impressive volume contains sections on a number of topics of interest to the water works engineer, such as fluid mechanics, structures and foundations, specifications and quantities, fire protection and town planning. In addition, there is included basic information on mathematics, statistics, the physical sciences, and mathematical and conversion tables.

The material is of course directly based upon British practice, which cannot be related to American conditions by so simple an operation as conversion of Imperial gallons to gallons U.S. measure. For example, in noting that the rate of domestic consumption of water is increasing toward the 30 gpm (Imp.) mark, the author of the water supply chapter, R. C. S. Walters, notes that "The higher consumptions per head in Scottish towns, as in the U.S.A., has not been satisfactorily explained." Likewise manufacturing specifications for such items as pipe differ in the two countries. On the other hand, an understanding of the other fellow's methods is one of the best aids to reevaluating and improving one's own.

Soil Engineering. *Merlin Grant Spangler. International Textbook Co., Scranton, Pa. (1951) 458 pp.; $6.50.*

Intended as a basic introduction to soil engineering, this book includes interesting sections on soil and ground water as they affect the engineering uses of soil. Earth dams, seepage, frost action in soils and flow nets typify the material of primary interest to the water supply fields, although to some extent much of the information on foundations and the bearing capacity and stress distribution of soils will interest those who want to know the possible complications involved, for example, in laying pipelines underground. A specific chapter on this topic, entitled "Underground Conduits," is also included.

(*Continued on page 80*)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Professional Services

Professional Services
(contd.)

Membership Changes

NEW MEMBERS

Applications received March 1 to March 31, 1952

Babcock, Russell H., Asst. Mgr., Utilities Industry Div., Foxboro Co., Foxboro, Mass. (Apr. '52) *MP*

Baker, V. W., *see* Wisconsin Rapids (Wis.) Waterworks & Lightning Com.

Baker Metropolitan Water & San. Dist., John Couper, Water Supt., 3019 Longfellow Pl., Denver 11, Colo. (Corp. M. Apr. '52) *MPR*

Bard, Thomas G., Civ. Engr., Berylwood Inv. Co., Somis, Calif. (Apr. '52)

Bauer, L. P., Mgr. & Treas., Maryville Deep Well Water Assn., Box 218, Georgetown, S.C. (Apr. '52)

Baum, Richard Edward, Supt. of Water Works, Mount Prospect, Ill. (Apr. '52) *M*

Baxter, Samuel S., Water Comr., 1103 City Hall Annex, Philadelphia 7, Pa. (Apr. '52) *MP*

Beauchemin, J. A., Beauchemin & Hurter, Cons. Engrs., 609 Drummond Bldg., Montreal, Que. (Jan. '52)

Berry, Theodore Victor, Comr., Greater Vancouver Water Dist., Vancouver, B.C. (Apr. '52) *M*

Bloxam, L. P., Supt., Elec. & Water Plant, 99 Market St., Bennettsville, S.C. (Apr. '52)

Burk, L. B., Water Works Supt., Eldorado, Tex. (Apr. '52) *M*

Caldwell, Creed, City Engr., Pine Bluff, Ark. (Apr. '52) *R*

Clancy, John H., Dist. Mgr., Johnston Pump Co., Box 1455 Burlington Station, Omaha, Neb. (Apr. '52)

Cote, Joseph Leon, Pres., L'Association des Proprietaires D'Aqueducs du Quebec, 175 Grande Allee, Quebec, Que. (Apr. '52) *MR*

Couper, John, *see* Baker Metropolitan Water & San. Dist. (Colo.)

Croker, Mark F., Water Comr., Water Dept., City Hall, Newton Centre, Mass. (Apr. '52) *M*

Denison, Edward S., *see* Latham Water Dist. (N.Y.)

Dickson, Donald Byron, Supt.-Acting Mgr., Sunshine Water Co., 11725 S. Carmenita Rd., Norwalk, Calif. (Apr. '52)

Dingwall, Douglas, Supt., Water Dist., 226 Main St., Presque Isle, Me. (Apr. '52)

Dunavant, William, Jr., Chief Engr., Fulton, Mo. (Apr. '52) *M*

Eagle, James Bryan, Accountant, Munic. Water Works, Little Rock, Ark. (Apr. '52) *M*

Gieser, Ralph H., Jr., Chief Chemist, Rheem Mfg. Co., 12000 E. Washington Blvd., Whittier, Calif. (Apr. '52) *PR*

Gossett, Orthy C., Field Repr., Winter Weiss Co., 5510 Wadsworth Blvd., Arvada, Calif. (Apr. '52) *MR*

Greenwood, William, City Engr., City Hall, Sarnia, Ont. (Apr. '52)

Groseclose, Herman C., Cons. Engr., 1915 N.W. 38th St., Oklahoma City, Okla. (Apr. '52) *MPR*

Hall, Charles Odell, Supt., Water Works, Rogers, Arkansas (Apr. '52) *MP*

Hall, M. Vell, Asst. Supt., Munic. Water System, Hot Springs, Ark. (Apr. '52) *R*

Harman, William Martin, Engr., Arkansas Inspection & Rating Bureau, 512 Hall Bldg., Little Rock, Ark. (Apr. '52) *M*

Harper, C. N., City Engr., Osage City, Kan. (Apr. '52)

Harwood, R. W., *see* Lindsay, P. K., Co.

Helmreich, Merwin Rea, Customer Service Section, Board of Water Supply, Honolulu, Hawaii (Apr. '52) *M*

Higginson, F. D., Supt., Public Utilities Com., Box 360, Picton, Ont. (Jan. '52)

(Continued on page 32)

(*Continued from page 30*)

Hixon, Albert E., Supt., Water Treatment, Town Hall, Cayce, S.C. (Apr. '52)

Humphries, J. B., Sr., Chief Operating Engr., Munic. Water System, Hot Springs, Ark. (Apr. '52) *P*

Huntsville Munic. Water Co., C. E. Shinn, Sr., Supt., Huntsville, Ark. (Corp. M. Apr. '52) *M*

Iles, William, Jr., Supt., Water Works, Board of Public Affairs, Box 667, Batavia, Ohio (Apr. '52) *MP*

Jackson, Ray Arthur, Asst. Engr., Constr. Div., Washington Suburban San. Com., Hyattsville, Md. (Apr. '52) *M*

Johnson, Wallace, Water Maint. Supt., Box 97, Huron, S.D. (Apr. '52)

Jones, David R., Distr. Supt., Shenango Valley Water Co., Sharon, Pa. (Apr. '52) *M*

Jones, Marvyn, *see* Walnut Ridge (Ark.) Water Works

Kennedy, A. P., *see* Toronto (Ont.) Township Waterworks

Kirwan, Kenneth K., Chief Engr., Centriline Corp., 140 Cedar St., New York 6, N.Y. (Apr. '52) *M*

Laing, William Henry, Jr., Water Supt., Chesterfield County Water Works, Chesterfield, Va. (Apr. '52) *M*

Latham Water Dist., Edward S. Denison, Supt., Town Hall, Newtonville, N.Y. (Mun. Sv. Sub. Apr. '52) *MPR*

Levin, Paul, Jr. San. Engr., Water Safety Control Section, 3300 E. Cheltenham Pl., Chicago 49, Ill. (Jr. M. Apr. '52) *P*

Lindsay, P. K., Co., R. W. Harwood, Sales Mgr., 97 Tileston St., Everett 49, Mass. (Assoc. M. Apr. '52)

Long, Robert F., *see* Long, V. Y., & Co.

Long, V. Y., & Co., Cons. Engrs., Robert F. Long, Partner, 1300 Colcord Bldg., Oklahoma City 2, Okla. (Corp. M. Apr. '52) *PR*

Longley, James Edward, Secy., Lock Joint Pipe Co., 150 Rutledge Ave., East Orange, N.J. (Apr. '52) *R*

Lybeer, Leopold, Gen. Foreman, Dept. of Water Supply, 3451 Orleans St., Detroit 7, Mich. (Apr. '52) *M*

Madison Board of Water Comrs., Bert E. Miller, Pres., City Hall, Madison 3, Wis. (Corp. M. Jan. '52) *M*

Meholick, George, Jr., City Mgr., City Hall, Wakefield, Mich. (Apr. '52) *MR*

Meinhertz, John, Water Supt., Sylvan Grove, Kan. (Apr. '52) *MR*

Meyer, Harlan W., City Engr., Box 97, Huron, S.D. (Apr. '52)

Miller, Bert E., *see* Madison (Wis.) Board of Water Comrs.

Nelson, Carl R., Accountant, Water Board, City Hall, Adrian, Mich. (Apr. '52) *M*

North, Harry S., Jr., Lab. Technician, Water Dept., Phoenix, Ariz. (Apr. '52) *P*

Parke, Harold Ralph, Chief Operator, Water Works, Sidney, Ohio (Apr.'52) *MP*

Pelton, H. L., Munic. Contractor, Box 341, Springdale, Ark. (Apr. '52) *MR*

Porter, Harry T., Sales Engr., Harry T. Porter Co., 1425 Union Central Bldg., Cincinnati 2, Ohio (Apr. '52)

Pratt, Willard S., Director of Public Works, City Hall, Newton, Mass. (Apr. '52) *M*

Rickards, J. A., *see* Washington (N.C.) Utilities Com.

Roberts, William J., Mgr., James B Clow & Sons, 1104 Riss Bldg., Kansas City, Mo. (Apr. '52)

Robinson, George J., Jr., Partner, Robinson Constr. Co., 711 Main St., Pine Bluff, Ark. (Apr. '52) *P*

Rodriguez, Luis M., Civ. Engr. & Architect, Falgueras No. 412, Cerro., Havana, Cuba (Apr. '52)

Ross, Alfred, Water Treatment & Sewage Disposal Supt., Box 97, Huron, S.D. (Apr. '52)

St. Peter City Council, E. V. Vinquist, Supt., Light & Water Dept., St. Peter, Minn. (Corp. M. Apr. '52) *M*

Sanderson, Marvin R., Branch Mgr., Layne Arkansas Co., Jonesboro, Ark. (Apr. '52) *R*

Schiavone, Ulderico M., City Engr., City Hall, Newton, Mass. (Apr. '52) *MR*

Schosser, Frank Alfred, Asst. Engr., Elec. & Water Eng. Dept., 34 S. Laura St., Jacksonville 2, Fla. (Apr. '52)

Schrauth, Amandus Joseph, Hydr. Field Engr., The Pitometer Co., 50 Church St., New York, N.Y.

(*Continued on page 34*)

(Continued from page 32)

Senkus, Walter F., Dist. Mgr., DeLaval Steam Turbine Co., 419 Stephenson Bldg., Detroit 2, Mich. (Apr. '52) *M*

Shinn, C. E., Sr., *see* Huntsville (Ark.) Munic. Water Co.

Sipe, John K., Water Supt., Mankato, Kan. (Apr. '52) *MR*

Smith, John W., Water Production Supervisor, Phoenix, Ariz. (Apr. '52)

Sockwell, Charles, *see* Sockwell Co.

Sockwell Co., Charles Sockwell, Jr., 156 Rogers St., N.E., Atlanta 6, Ga. (Assoc. M. Apr. '52)

Stearns, Fred M., Utilities Supt., Dighton, Kan. (Apr. '52) *MR*

Stephenson, C. A., Chief Engr., Arkansas Inspection & Rating Bureau, 512 Hall Bldg., Little Rock, Ark. (Apr. '52)

Swartz, Samuel O., Bacteriologist, Water Div., Metropolitan Dist. Com., 20 Somerset St., Boston, Mass. (Apr. '52) *MP*

Tanck, Ralph E., Supt., Sewer & Water Dept., Ness City, Kan. (Apr. '52) *MR*

Toronto Township Waterworks, A. P. Kennedy, Engr., Cooksville, Ont. (Corp. M. Apr. '52)

Valente, George A., Borough Engr., Caldwell, N.J. (Apr. '52)

Vinquist, E. V., *see* St. Peter (Minn.) City Council

Walnut Ridge Water Works, Marvyn Jones, Mgr., Walnut Ridge, Ark. (Corp. M. Apr. '52) *MP*

Washington Utilities Com., J. A. Rickards, Supt., Washington, N.C. (Corp. M. Apr. '52) *MPR*

Wisconsin Rapids Waterworks & Lighting Com., V. W. Baker, Supt., Wisconsin Rapids, Wis. (Corp. M. Apr. '52) *MPR*

Witte, M. A., Testing Engr., Oklahoma Testing Labs., 310 N. Klein St., Oklahoma City, Okla. (Apr. '52) *MP*

Wood, Warren Frederick, Haddon Supt. of Public Works, 8 Reeves Ave., Westmont, N.J. (Apr. '52) *M*

Wooster, Harry M., Engr., Shenango Valley Water Co., Sharon, Pa. (Apr. '52) *M*

Wyshank, Leo T., Asst. San. Engr., Water Safety Control Section, Div. of Water Purif., 3300 E. Cheltenham Pl., Chicago 49, Ill. (Apr. '52) *MP*

REINSTATEMENTS

Ammons, Delton E., Dist. Supt., Southern California Water Co., 10426 S. Normandie Ave., Los Angeles 44, Calif. (Apr. '49)

Clemens, Levon, Water Works Supt., Kingsford Corp., R.R. 2, Laporte, Ind. (Oct. '48)

Clift, Mortimer A., Sales Agent, 815 Berkley Rd., Indianapolis 8, Ind. (Oct. '49)

Dan River Mills, Inc., H. C. Jones, Supt., Filtration, Danville, Va. (Corp. M. Apr. '48)

Daykin, Jason N., Supt., City Water Dept., Taylorville, Ill. (July '48)

Health, Donald M., Service Engr., Morton Salt Co., 801 S. 11th St., Lafayette, Ind. (Jan. '50) *P*

Horner, J. L., City Mgr., Carthage, Tex. (Oct. '39) *M*

Joliet, City of, Joseph F. Whalen, Comr. of Public Property, City Hall, Joliet, Ill. (Corp. M. Oct. '41)

Jones, H. C., *see* Dan River Mills, Inc.

Letbetter, W. M., Supt., Water & Sewer Improvement Dist., 122 W. 2nd St., Corning, Ark. (Jan. '49)

Lindeen, Leonard F., Foreman, Distr. System & Meter Div., City Water Dept., Box 143, Wilmette, Ill. (Jan. '46)

Rauscher, John F., Engr., Water Dept., City Hall, Tucson, Ariz. (Oct. '47)

Sells, James, Sales Engr., Rockwell Mfg. Co., Box 290, Alexandria, La. (Jan. '50)

Whalen, Joseph F., *see* Joliet (Ill.)

LOSSES

Deaths

Berigtold, Charles N., Jr. Assoc. Engr., Bureau of Water Supply, 8 W. 2nd Ave., Baltimore 25, Md. (Oct. '49) *M*

Gilbert, Gordon M., Comr., Greater Vancouver Water Dist., Sun Bldg., Vancouver, B.C. (Jan. '49)

Hawley, John J., Sales Service, Hersey Mfg. Co., 4405 S. Calhoun St., Fort Wayne 6, Ind. (Apr. '43)

Hayes, Joseph A., Sales Engr., Neptune Meter Co., 2701 Chestnut St., Camp Hill, Pa. (Apr. '49)

(Continued on page 36)

(*Continued from page 34*)

Murray, Joseph J., Water Comr., Water Dept., 1000 Commonwealth Ave., Newton Centre 59, Mass. (Oct. '39) *MR*

Winkley, John M., Supt., Water Works, Monon, Ind. (Apr. '46) *M*

Resignations

Allison, William R., *see* Washington (Ind.) Water Works

Moon, Gerald R., Water Treatment Technician, Blackford Window Glass Co., Vincennes, Ind. (July '51) *P*

Washington Water Works, William R. Allison, Office Mgr., City Hall, Washington, Ind. (Corp. M. Apr. '51) *M*

CHANGES IN ADDRESS

Changes received between March 5, and April 5, 1952

Anderson, Frank W., Gen. Purchasing Agent, Metropolitan Utilities Dist., 18th & Harney Sts., Omaha 2, Neb. (Apr. '51)

Arismendi, Carmelo, Instituto Nacional de Obras Sanitarias, Reducto a Clorieta 88, Caracas, Venezuela (Oct. '51) *MP*

Banks, Harvey O., 431 Ross Way, Sacramento 21, Calif. (Jan. '41) *PR*

Bass, J. H., Infilco Inc., Claypool Hotel, Indianapolis, Ind. (Oct. '46)

Beamer, Norman H., Dist. Chemist, U.S. Geological Survey, 1302 Customhouse, 2nd & Chestnut Sts., Philadelphia, Pa. (Jan. '49) *R*

Bosch, A., *see* National Automatic Sprinkler & Fire Control Assn.

Burba, Foster S., 644 Sizeler St., New Orleans, La. (Jan. '49) *MPR*

Carroll, John T., Asst. Dist. Mgr., Worthington Corp., LaSalle Hotel, South Bend, Ind. (July '25)

Chandler, Sidney W., Civ. Engr., Biggs, Weir & Chadler, 336 Meadowbrook Rd., Jackson, Miss. (Jan. '51) *P*

Coleman, Richard D., Dept. of Industrial Health, Harvard School of Public Health, 55 Shattuck St., Boston 15, Mass. (Apr. '51) *P*

Dunstan, James, 1043 Monroe St., Wenatchee, Wash. (Apr. '38) *MP*

Foote, Herbert Branch, Div. Environmental San., State Board of Health, Helena, Mont. (Aug. '23) *Director '31–'33. Fuller Award '42. MP*

Garland, Chesley F., 3243 E. Linden St., Tucson, Ariz. (Jan. '42) *MPR*

Godshall, D. A., Sr., Civ. Engr., 335 E St., Oxnard, Calif. (Apr. '51)

Grune, Werner N., Public Health Engr., Div. of Water Pollution Control, U.S. Public Health Service, 12 Lynn Court, Rutherford, N.J. (Oct. '49) *MPR*

Harper, L. E., Pres., Omega Machine Co., 345 Harris Ave., Providence 1, R.I. (July '37) *P*

Hauck, Charles F., Engr., Chem. Plants Div., Blaw Knox Constr. Co., 930 Duquesne Way, Pittsburgh 30, Pa. (Apr. '42) *MPR*

Heiberg, A. S., *see* Willmar (Minn.) Water & Light Dept.

Hubbard, E. C., Exec. Secy., State Stream San. Com., Box 2091, Raleigh, N.C. (Oct. '43) *MPR*

Ilion Board of Water Comrs., C. Gordon Rahm, Mgr., Ilion, N.Y. (Mun. Sv. Sub. Mar. '24) *MPR*

Karpen, Raymond J., MSC, 4330 Marlborough St., San Diego, Calif. (Jan. '46) *P*

Klaus, Fred J., 1311—15th St., Sacramento, Calif. (Oct. '15) *M*

Koch, Alwin G., 6536—102nd Pl., N.E., Kirkland, Wash. (July '50) *PR*

Kochtitzky, Oscar W., Jr., Health & Safety Div., Tennessee Valley Authority, Chattanooga, Tenn. (July '49) *PR*

LaMarre, Rene J., Route 2, Box 354, Pompano Beach, Fla. (Jan. '35) *Director '47–'50.*

Lamb, James C., III, 284 Westgate W., Cambridge, Mass. (Oct. '49)

Langworthy, Virgil W., 136 Sylvan Knoll Rd., Stamford, Conn. (Jan. '50)

Lau, Curtis W., Sales Engr., 314 Old Garden Lane, York, Pa. (Oct. '50)

Lozada C. L. A., Carrera 19, No. 124, Barquisimeto, Venezuela (Jan. '41)

MacLean, Gordon E., Sales & Service, Dearborn Chem. Co., 3503 Rainbow Pl., Nashville 4, Tenn. (Oct. '49)

Macomber, Ronald Gibbs, Mutual Security Agency, APO 928, c/o Postmaster, San Francisco, Calif. (Jan. '48) *PR*

(*Continued on page 38*)

(Continued from page 36)

McBride, George A., Sales Engr., Infilco Inc., Van Orman Hotel, Fort Wayne, Ind. (July '41) *P*

McCall, Robert George, MSC, Army Environmental Health Lab., Army Chemical Center, Md. (Jan. '39) *P*

McKimmon, William S., Chief, Eng. Section, San Eng. Div., State Board of Health, Box 2091, Raleigh, N.C. (Jan. '50)

Moloney, John Irwin Thomas, 1627 Virginia Dr., Urbana, Ill. (Jr. M. Jan. '52) *P*

Munroe, Walter C., Chief Engr., Anne Arundel County San. Com., Glen Burnie, Md. (Jan. '24) *P*

Moore, Carl C., Dist. Mgr., Pittsburgh Equitable Meter Div., Rockwell Mfg. Co., 600 Grant St., Pittsburgh, Pa. (Apr. '39)

National Automatic Sprinkler & Fire Control Assn., A. Bosch, Secy.-Treas., 205 E. 42nd St., New York 17, N.Y. (Corp. M. Jan. '42)

Nesbitt, John B., 250 S. Gill St., State College, Pa. (Jr. M. Jan. '49) *MPR*

Olsen, Carl Stanford, 10903 S.E. 25th St., Bellevue, Wash. (Oct. '48) *MPR*

Paessler, Alfred H., Exec. Secy., State Water Control Board, 415 W. Franklin St., Richmond 20, Va. (Jan. '49) *R*

Palm, Millard Blaine, AO 2239595, 5th Medical Squadron, Travis Air Force Base, Fairfield, Calif. (Jr. M. Oct. '51)

Pell, William I., Claymont Steel Corp., Claymont, Del. (July '45) *R*

Pitman, Ike W., Salesman, Neptune Meter Co., 254 Spring St., N.W., Atlanta 3, Ga. (July '49)

Pittman, William G., *see* Pittman, William G., Co., Inc.

Pittman, William G., Co., Inc., William G. Pittman, 61 Voorhis Lane, Hackensack, N.J. (Assoc. M. Oct. '49)

Proske, H. O., Sales Engr., Rockwell Mfg. Co., 1249 Burlington Ave., North Kansas City, Mo. (Jan. '50)

Rahm, C. Gordon, *see* Ilion (N.Y.) Board of Water Comrs.

Schwada, Joseph P., 2736 N. Sholes Ave., Milwaukee, Wis. (May '24) *Fuller Award '44. Director '46–'49. Diven Medal '49. P*

Smith, Philip W., Field Supt., Dept. of Water & Power, 3426 Rowena Ave., Los Angeles 27, Calif. (Oct. '43) *M*

Stead, John E., Pres., Indian Head Water Co., Inc., 1427 E. Lafayette St., Tallahassee, Fla. (Jan. '51) *M*

Stone, Raymond V., Jr., Research San. Engr., Univ. of California, 2180 Milvia St., Berkeley, Calif. (Apr. '50)

Striger, R. M., Cavallo & Wise, Engrs., 203 Barnett Maddan Bldg., Jackson, Miss. (Oct. '45) *P*

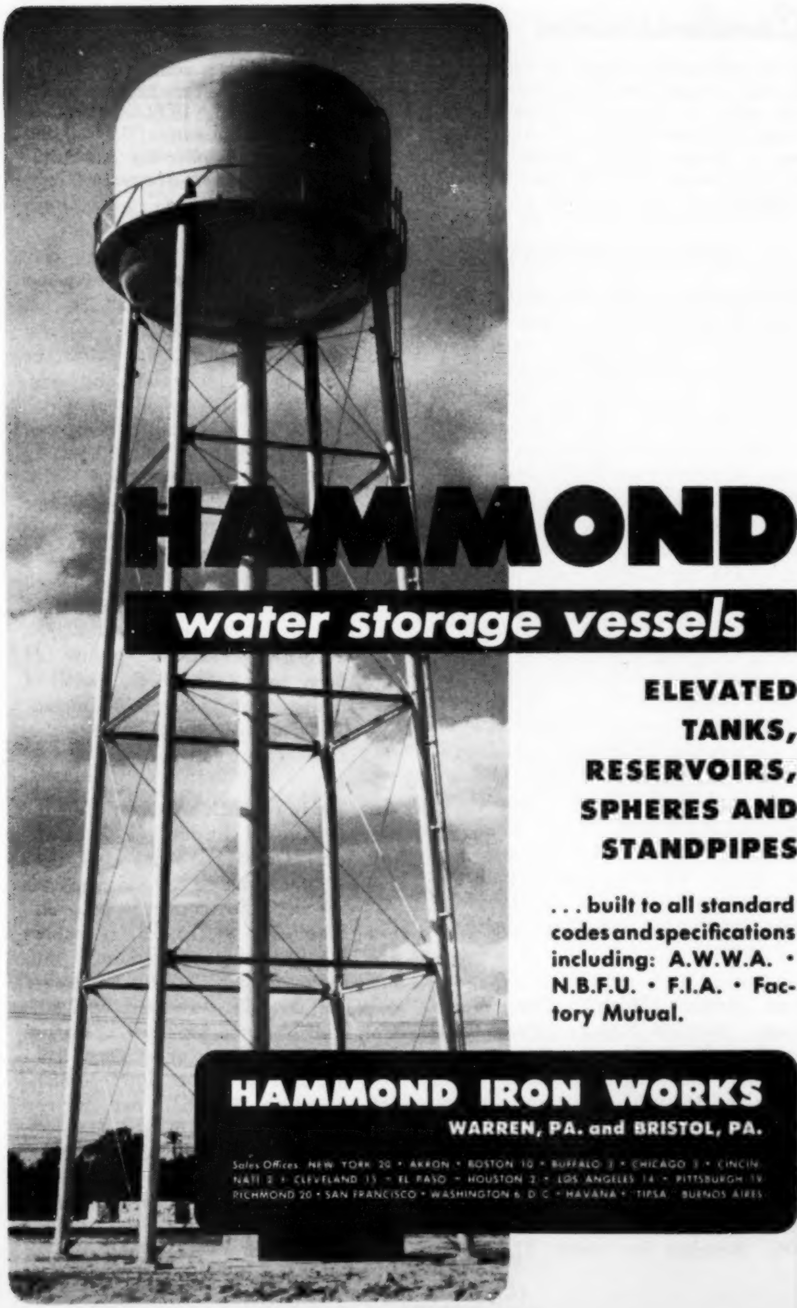
Thoits, Edward D., Sales Div., The Dorr Co., Inc., Box 21, Palo Alto, Calif. (Jan. '51)

Vetromile, Daniel S., Sales Engr., 4 Alder Dr., Ramsey, N.J. (Oct. '49)

Whittaker, Harold Arthur, National Research Council, Div. of Medical Sciences, 2101 Constitution Ave., Washington 25, D.C. (June '13)

Willmar Water & Light Dept., A. S. Heiberg, Supt., Box 466, Willmar, Minn. (Corp. M. July '50)

Yegen, William, Supt., Filtration Plant, Star Route No. 2, Bismarck, N.D. (June '27) *P*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Condensation

Key: In the reference to the publication in which the abstracted article appears, 39:473 (May '47) indicates volume 39, page 473, issue dated May 1947. If the publication is paged by the issue, 39:5:1 (May '47) indicates volume 39, number 5, page 1, issue dated May 1947. Abbreviations following an abstract indicate that it was taken, by permission, from one of the following periodicals: *B.H.—Bulletin of Hygiene* (*Great Britain*); *C.A.—Chemical Abstracts; Corr.—Corrosion; I.M.—Institute of Metals* (*Great Britain*); *P.H.E.A.—Public Health Engineering Abstracts; S.I.W.—Sewage and Industrial Wastes; W.P.A.—Water Pollution Abstracts* (*Great Britain*).

ANNUAL REPORTS

Metropolitan Water Board (Gt. Br.). 47th Annual Report (Year Ending Mar. 31, 1950). 128 pp. In the annual report of the Metropolitan Water Board for the year ended 31st March 1950 information is given on the sources, distribution, storage, and treatment of water supplies for the Metropolitan district. The standard of purity of the board's supply was higher during 1949–50 than in previous years. Of 13,154 samples of water taken before distr., 99.9% contained no *Esch. coli* per 100 ml; 99.3% of samples taken from consumer's premises were free from *Esch. coli.* At all filtration plants except one, water is now treated by superchlorination resulting in an increase in bacteriological quality and in a reduction of 30% in color. In an appendix data are given of the amts. of water taken from the rivers Lee and Thames and from wells and springs in the area, on rainfall and stream flow in Thames and Lee Valleys, and on bact. and chem. quality of the water.

Ann Arbor (Mich.) City Water Dept. Annual Report (Year Ending Jan. 31, 1950). Total assets $3,460,805, including $2,091,383 plant assets and $670,501 constr. in progress, total liabilities $1,274,980, of which $1,241,000 bonds outstanding, municipal equity $2,185,825. Total receipts $1,736,816, of which bond issue $1,102,173, total disbursements $1,575,306. Receipts for constr. $1,236,486, disbursed $1,172,059. Operating revenue $341,863, operating expense $302,595, including depn. $50,374, net operating income $39,268, interest $18,958, net income $32,001. Cost per mil gal pumped, softened, and sold, resp., $40.60, $43.07, $47.40 (of which about 50% for chemicals), compared to 10-yr avgs. $31.47, $32.61, $37.26, resp. Power cost 0.93¢ per kwhr, $10.72 per mil gal pumped. Water unaccounted for 14.35%, 5.6% less than in 1948. Avg. consumption 6.53 mgd., 163 gpcd., max. 10.4 mgd. Well water comprised 66% of supply, river water 34%. Estd. pop. served 40,000, meters 9378, hydrants 538, mains 121 mi. New mains 13,418 ft, mostly 4 and 6″ avg. cost $3.89 per ft, including 1136′ of 12″ at $8.56 per ft.—*R. E. Thompson.*

Ann Arbor (Mich.) City Water Dept. Annual Report (Year Ending Jan. 31, 1951). Total assets $3,514,365, including $2,080,585 plant assets and $1,100,584 constr. in progress, total liabilities $1,253,672, of which $1,229,000 bonds outstanding, municipal equity $2,260,692. Total receipts $1,066,775, disbursed $847,403. Operating revenue $367,859, operating expense $300,957, including depn. $47,984, net income before interest $84,160, interest $31,463, net income $52,698. Cost per mil gal pumped, softened, and sold, resp., $37.93, $41,07, and $45.59, compared to 11-yr avgs. $32.17, $33.49, and $38.15, resp. Power cost per kwhr 0.92¢, per mil gal pumped $10.42. Water unaccounted

(*Continued on page 42*)

(*Continued from page 40*)

for 16.84%. Avg. consumption 6.74 mgd, 168 gpcd, max. 10.97 mgd. Well water 68% of supply, river water 32%. Estd. pop. served 40,000, meters 9677, hydrants 550, mains 130 mi. Cost of 12,852′ new mains laid, mostly 6″, $3.16 per ft.—*R. E. Thompson.*

Brockville (Ont.) Public Utilities Com. Annual Report (1950). Pumpage 1126 mil gal, 759 by domestic consumers. Hydrants 188, valves 404, services 3662, 62 metered. Revenue $53,596, expenditures $48,-687, profit $4,909. Domestic rates $6.72 per yr for up to 3 rooms, $2.40 per room for addnl. rooms up to 6, $1 per room over 6. Outside town limits 25% higher. Meter rates 8 to 25¢ per 1000 gal, with 10% discount for prompt payment; min. monthly bill $1.20.—*R. E. Thompson.*

Dubuque (Ia.) Water Dept. Annual Report (Year Ending March 31, 1951). Supply dates from 1870, municipally owned since 1900. Source—eight artesian wells, 1300 to 1781′ deep, 6 to 16″ diam., and mine tunnel known as "The Level." Storage: three reservoirs, 10.3 mil gal total capac.; two elevated tanks, 0.5 and 0.75 mil gal capac.; and 0.6-mil gal standpipe. Pop. 49,527, mains 130 mi, valves 2508, hydrants 990, meters 11,-246. Rates (21% increase) 7 to 27¢ per 100 cu ft; monthly min. charge $0.82 to $15 according to meter size; sprinkling system charge $25 to $130/yr, depending upon number of heads. Net profit $89,023, 4% of plant investment. Capital improvements ($172,-459) included main extensions and elevated tank. Revenue bonds, $100,-000, sold to complete distr. system improvements; sinking fund for retirement in 4 yr. This is only bonded indebtedness. Net value of water plant $2,312,616. Static water level drop of 80′ in past 12 yr has introduced two

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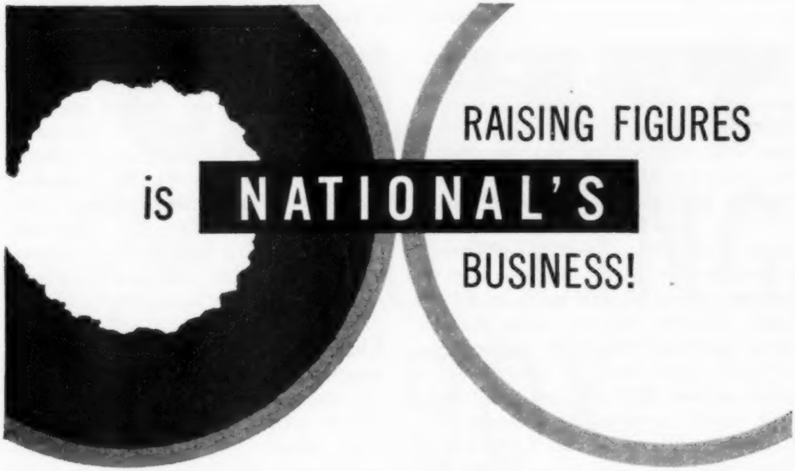
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(*Continued from page 42*)

new problems: [1] red water in outlying mains, [2] 12% decrease in yield per ft drawdown. Further deep well development not practicable. If shallow well exploration not successful, recourse to Mississippi R. possible. System in Class 1 of National Board of Fire Underwriters. Free water, absence of hydrant rental, and contribution to city admin. officers' salaries save taxpayers $80,000 annually. Revenue $284,150, 96.76% from metered sales, operation $125,525, maintenance $27,592, depn. $41,385, interest $625, total expense $195,127, net profit $89,023, city's proprietary interest $2,312,616. Fixed assets $3,026,042, depn. $961,094, total value $2,064,948. Power costs at Eagle Point Station (lift 224′) 2.9¢, West Third St. Station (200′) 1.11¢, Level Station (250′) 1.99¢ per 1000 gal. Avg. consumption 4.3 mgd, 87 gpcd. Cost of power $25.04, operating profit $38.84, total receipts $131.62 per mil gal pumped. Total cost per 1000 gal consumed 12.44¢. Water sold 73.28% of pumpage, accounted for 84.15%, supplied free 4.88%, underregistration 2.2%, unaccounted for 13.65%. Bacterial count (48 hr) on samples from various locations 0 to 26, gas formers absent in 10 ml.—*R. E. Thompson.*

Elmira (N.Y.) Water Board. Annual Report (1950). History of supply since inception in 1858 reviewed. Jewell rapid sand filter plant of 18 units installed in 1896, following typhoid outbreak, still operating satisfactorily. Chlorination commenced 1909. Municipal ownership since 1915. About one-third of supply derived from Hoffman Creek Reservoir

(*Continued on page 46*)

(Continued from page 44)

(150 mil gal capac.), 1 mi north of plant. Tributary watershed 5 sq mi, portion owned by board reforested with evergreen trees. Balance of supply from Chemung R., drainage area of 2050 sq mi. Purif. process including prechlorination, alum coagulation, 8 to 10 hr settlement, rapid sand filtration (24 units), and final chlorammoniation treatment. Water pumped from 3.5-mil gal low-service reservoir, to which water flows from filters, to spray nozzle aerators around 5-mil gal high-service reservoir. Latter provides pressure in excess of 70 psi in downtown area. Covered balancing reservoir (1.5 mil gal) south of river. Metering 100%. Anals. given for both supplies. Hoffman Creek: color reduced from 40 ppm to nil, turbidity from 50 to nil, bacterial count per ml, gelatin, 48 hr 20 C, 550 to 1, *Esch. coli* to 0 (standard sample), total solids 105 to 85 ppm, hardness 61.8 ppm. Chemung R.: color from 8 to 0, turbidity 5 to 0, total solids 230 to 185, bacterial count 150 to 1, *Esch. coli* to 0, hardness 179. Rates 8 to 35¢ per 100 cu ft, min. charge $2.80 per quarter; outside city 9 to 42¢, min. $3.36; all subject to 10% discount for prompt payment. Meters 17,316, hydrants 874, mains 173 mi, services 15,762. Main costs per ft: ¾ to 2″ $2.26, 6″ $3.78, 8″ $4.35, 10″ $5.62, 12″ $6.06. Total assets $3,979,681, property value $3,662,649, depn. reserve $1,064,361. Operating revenue $402,735, operating costs $313,422, including taxes $17,743 and depn. $54,212, operating income $89,313, nonoperating income $3,448, net income $92,761. Estd. pop. served 65,000, avg. consumption 7.04 mgd (6.8% decrease), 108 gpcd. Pptn. 43.8″, 20-yr avg. 33.6″.—*R. E. Thompson.*

London (Ont.) Public Utilities Com. Annual Report (1950). Estd. pop. 106,271, avg. pumpage 9.86 mgd, 92.8 gpcd. Supply chiefly from springs, supplemented by river water. Services metered 100%, percentage of pumpage metered 85.4. Meters 25,830, gate valves 2221, hydrants 1357. Revenue $666,385, including fire service $23,508, expenses $356,548, operating surplus $309,837. Disposition of latter: interest $157, sinking fund and installments $3,098, constr. out of revenue $251,157, surplus $55,425.—*R. E. Thompson.*

New Orleans (La.) Sewage and Water Board. Semiannual Report (Dec. 31, 1950). Purif. plant capac. increased to 112 mgd in '32, operated at rates in excess of 150 mgd. Plans and studies for extension to 200 mgd. Power plant extensions also in progress. Operation and maintenance costs increased 5% during '50. Two plants, Carrollton serving New Orleans, and Algiers. Avg. high-lift pumpage, New Orleans, 84.39 mgd (max. hourly rate 122), 68% by elec. centrifugals, balance by vertical triple expansion engines. Cost of high-lift pumping $5.38 per mil gal. At Carrollton plant, 97.2 mgd Mississippi R. treated (max. day 129.6): all suspended matter (106,577 tons) removed by coagulation and filtration and 44.6% of hardness by lime treatment. Purif. process includes lime treatment followed by primary sedimentation (98.2% of turbidity removed), flocculation with $FeSO_4$, or equal parts of $FeSO_4$ and alum, followed by sedimentation, disinfection with chloramine, ammonium sulfate, and 95% of Cl added prior to filtration and 5% after. Cl residual at plant 0.5 ppm and throughout distr. system 0.46. Wash water avgd. 1.09% of water filtered, and its cost, at $13.19 per mil gal, was 15¢ per mil gal of water filtered. Cost of water used for washing basins 10¢ per mil gal treated, total cost 71¢ (539,000 cu yd wet mud removed). Gross cost per mil gal

(Continued on page 48)

(Continued from page 46)

treated and pumped \$18.02, of which purif. cost \$9.56. At Algiers plant, operation paralleled that at Carrollton, avg. treated 3.88 mgd and cost of purif. \$29.13 per mil gal. At Carrollton, turbidity reduced from 667 ppm. (max. 1570) to 0.35 (max. 4), hardness from 121 (max. 182) to 67 (max. 82), 37 C agar count from 71,335 (max. 2,000,000) to 3 (max. 103). Chem. dosages, avg. and max., resp.: lime 63.2 and 84.2 ppm, $FeSO_4$ 6.2 and 10.8 ppm, Cl. 9.46 and 10.66 lb per mil gal, ammonium sulfate 10.96 and 13.13 lb per mil gal. Settled water turbidity, avg. 12, max. 30 ppm, avg. filter run 160, max. 380 hr, avg. rate of filtration 108 mgd. Avg. and max. chem. dosages, resp., at Algiers: lime 63.2 and 77.0 ppm, $FeSO_4$ 96 and 15.3 ppm, Cl 285.9 and 330.4 ppm, ammonium sulfate 337.3 and 354.4 ppm. Avg. turbidity of settled water 7 ppm, max. 9, avg. filter run 212 hr, max. 260, wash water 1.0%, avg. rate of filtration 98 mgd. Avg. pH of river water 8.1, of treated water 10.1. Monthly avg. water temp. at Carrollton 46 to 84 F, during 36-yr period max. 91.4 F, avg. 66.3 F. Max. chlorides at Carrollton 45 ppm, Algiers 56. Mains 950 mi, valves 8237, hydrants 9081, services 106,522, meters 104,496. Rainfall 40.65", 57-yr avg. 57.15". Constr. and extensions fund (derived from taxes): income \$3,192,063, disbursements \$4,-001,060. Water works-sewerage, operating, and maintenance fund: income \$3,345,526, of which \$2,823,101 from sale of water, disbursements \$3,380,-400, of which \$1,740,716 for waterworks operation and maintenance. Free water 3806 mil gal. Of water pumped, 54.7% sold, free water from

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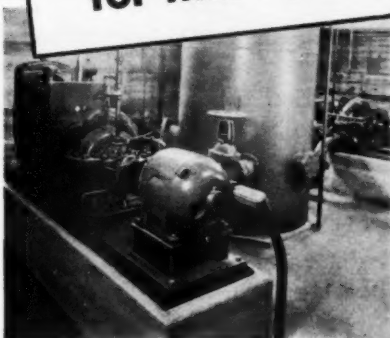
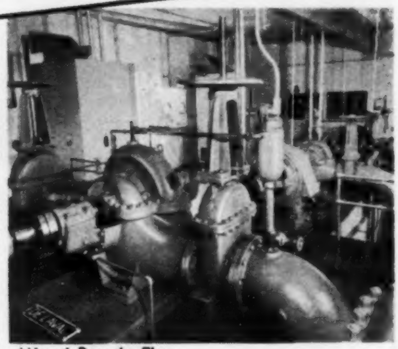
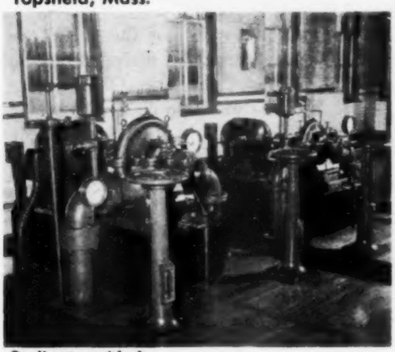


Topsfield, Mass.



Miami Beach, Fla.



Burlington, N. J.

Lynn, Mass.

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city departments and charitable institutions 5.6%, free water for sewerage 4.1%, allowance for leakage 0.6%, unmetered free use (fire fighting, street and sewer flushing, underregistration, leakage, etc.) 35%.—*R. E. Thompson.*

STREAM POLLUTION CONTROL

Ohio River Drainage Basin. Federal Security Agency. Water Pollution Series No. 12, 1951. The Ohio R. Basin has a total drainage area of approx. 163,000 sq mi exclusive of the Tennessee R. Basin and drains part of N.Y., Pa., N.C., W.Va., Md., Va., Ohio, Ky., Ind., Ill. and Tenn. Although the Tennessee R. drains into the Ohio R., the Tennessee Basin is considered separately and is not included in this report.

The major portion of the area covered by this report is included in the district covered by the Ohio River Water Sanitation Compact formed by eight of the states in the basin. This report includes the portions of N.C., Md., and Tenn. which drain into the Ohio R. drainage system (exclusive of the Tennessee R. drainage system) but which do not form part of the sanitation compact district. The portions of Va. and Ky. which drain to the Tennessee R. Basin are included in the sanitation compact district but are excluded from this report. In view of the association between the sanitation compact and the U.S. Public Health Service in matters related to poln. abatement in the basin, the above comparison of areas serves to avoid misinterpretation in correlating data pertinent to the sanitation compact district with the basin data.

The upper portion of the basin is hilly to mountainous, the southwestern part has rolling hills and wide valleys, and the northwestern portion is level or gently rolling.

The location of the basin and its natural resources have combined to

(Continued on page 52)

(*Continued from page 50*)

make the area one of the most important agricultural and industrial sections of the nation. The values of the annual agricultural and industrial products are approx. 9 and 15%, respectively, of the total annual value of these products in the nation. Practically all types of industry exist in the basin. The 1940 pop. of the basin was approx. 16,324,000, and preliminary 1950 census figures indicate approx. 17,738,000.

Water uses in the basin include public and industrial water supplies, navigation, power development, fish and wildlife propagation, recreational purposes, irrig., and final waste disposal. In some areas ground water is available for small to moderately large demands. The demands for municipal and industrial supplies are increasing and availability of water for these uses is vital for the continued growth of this basin. In other areas use of surface waters is required. Surface waters are used for large municipal and industrial supplies throughout the basin. Navigation developments provide one of the largest and busiest systems in the world. Favorable locations exist for large scale development of hydroelectric power. Water uses for recreation have always been important in the basin and are becoming more important. Agricultural developments require a widespread supply for livestock water supplies, and irrig. requirements may become increasingly important. The streams of the basin will continue to receive the final effluent from municipalities and industries. Conflicts between water uses are increasing as pop. and industrial developments increase. One of the most important conflicts is that between the use of the streams for waste disposal and other uses.

Polluting wastes are discharged from 2,104 centers of pop. and from 1,264 industries through sewers not connected to municipal systems. Of the 2,104 municipal sources of poln., 512 had 1940 pop. exceeding 2,500; 557 had pop. from 1,001 to 2,500; 545 had pop. from 501 to 1,000; and 490 had pop. of 500 or less. A large number of the listed industries have relatively small poln. loads. Many small canneries are included in the list and, in numerous instances, poln. effects are minimized by lagooning wastes during critical periods and discharging them during high-flow low-temperature conditions. The total organic waste load discharged to the streams from 1,204 municipal sources of poln. is estd. at approx. 8,760,000 equiv. pop. and the organic waste load from 383 industrial sources is estd. at 4,525,000 equiv. pop. The waste load discharged from 900 municipalities and the organic

(*Continued on page 54*)

(*Continued from page 52*)

waste load from 326 industrial sources have not been detd. Inorganic waste, not measured by pop. equiv., is discharged from 635 industrial plants, 128 of which also discharge organic wastes. The type of waste from 48 industrial plants has not been detd. The reported total organic waste load discharged to the streams is estd. at only approx. 70% of the total load actually discharged. Inorganic wastes from the above industries include acid from steel plants, toxic metal wastes, phenols, and other taste and odor producing substances from coke and chemical plants, brines, sulfates, color producing substances, and silt from coal washing operations.

Acid mine drainage is the greatest single poln. problem in the basin and constitutes the greatest unknown with respect to satisfactory and economical means for control and abatement. The sources of mine drainage have not been listed in the report. Both active and abandoned coal mines contribute poln. to the streams and the number of sources, particularly abandoned mines, are numerous. Moreover, the number is increasing and the potential magnitude of the problem can be visualized from the fact that about 75% of the nation's coal is supplied from the basin, only about 7% of the coal reserves have been touched, and approx. 55% of the acid load comes from abandoned mines.

Damages from stream poln. occur to all water uses in the basin. These damages may be the result of bacterial pollution, deoxygenation, toxicity, acid conditions, chloride content, or the presence of color-, odor-, or taste-producing substances. Stream poln. in

(*Continued on page 58*)

Why cast iron pipe

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this river basin results in damages which require relocation of water supplies, necessitate additions to water treatment plants, cause additional cost of water treatment, create hazards to public health from bacterial poln., preclude use of waters for water supplies and recreation purposes, destroy or inhibit fish and other desirable aquatic life, and cause corrosion damage to equip. and structures. Many of the poln. problems are interstate in character and became of such importance that, in 1948, eight states in the basin joined together in a compact for a cooperative approach to the solution of the problems. The damages from poln. are both tangible and intangible and no est. of their magnitude in terms of dollars is feasible at the present time. Undoubtedly, the overall benefits which will result from poln. abatement demand that such abatement be attained.

Of the 2104 sources of municipal poln., 224 have primary treatment and 342 have secondary treatment. Approx. 38% of the pop. served by sewers is connected to these treatment plants but at least 159 of the plants are inadequate in capac. Treatment facilities of some type have been reported for 451 of the 1264 industries listed as sources of poln. The status of treatment at 277 locations has not been detd. At least 164 of the industrial waste treatment plants are inadequate in capac. Ninety-eight municipal and 77 industrial treatment facilities are reported to be unsatisfactorily operated.

Thirty-three municipal treatment plants were constructed or improved between 1946 and 1949. Although the progress indicated by the construction of these plants in the 4-yr postwar period appears inadequate, it must be realized that the 2-yr period following the war was one of unsettled labor and material markets. Many projects are

(Continued on page 60)

(*Continued from page 58*)

now in the planning, financing, and constr. stages. There were 160 industrial waste treatment plants constr. in this period, a large percentage of which were constructed at canneries.

On the basis of existing data, new treatment plants are required for 911 municipal sources of poln. and 539 industrial sources. Enlargements of, or additions to, existing facilities are required for 132 municipalities and 108 industries. Existing treatment facilities for 39 municipalities and 33 industries should be replaced. Requirements have not been detd. for 695 municipal and 353 industrial sources of poln. At this time 327 municipal and 231 industrial sources of poln. require no project for poln. abatement. Corrective measures have been initiated for 495 municipal and 212 industrial sources. Of the 1777 municipalities listed as requiring projects for poln. abatement or listed as sources of poln. for which requirements had not been detd., there are 408 which had 1940 pops. in excess of 2500; 451 which had pops. from 1001 to 2500; 488 which had pops. from 501 to 1000; and 430 which had pops. of 500 or less.

Continued attention should be given to securing good operation of municipal and industrial waste treatment plants.

Low-flow regulation, where it is feasible, is a valuable supplement to waste treatment plants in reducing the damaging effects of poln. Flow regulation in addition to that which has been provided is desirable.

Research is necessary to devise and perfect practical and economical methods of treatment of industrial wastes

(*Continued on page 62*)

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(Continued from page 60)

which are not susceptible to known effective methods of treatment or for which present methods are costly. Acid mine drainage, an important source of poln. in this basin, is an example of industrial waste for which no satisfactory or economical treatment method is known, at present.

Effective state legislation is necessary to promote poln. abatement. Such legislation has been obtained in some states recently. Other states are proposing addnl. legislation and possibly some of the states will find it necessary to extend or supplement existing laws in the future.

Restrictive and other legislation affecting the financing of poln. abatement programs requires consideration in some instances.

Continuation and expansion of informational programs, designed to inform the public fully of the damages accruing from poln. and the benefits to be derived from poln. abatement, are a necessary part of an effective poln. abatement program.

On the basis of available data, which are incomplete at this time, and assuming the average cost index for 1949, the approx. capital cost of facilities required for poln. abatement in the basin is estd. at $350,000,000 to $400,000,000, plus addnl. requirements for abatement of poln. resulting from inorganic wastes and acid mine drainage.

Stream Pollution in the Oklahoma City Area. G. B. TREAT. Southwest W.W. Jour., **31**:12:17 ('50). In addition to problems of poln. usually encountered by cities, Oklahoma City has one serious problem caused by the extensive development of nearby oil fields which discharge waste oil and brine. There are often large quantities of oil in the sewage at the South Side treatment plant and this is skimmed off and burned or disposed of in lagoons. The commonest method for disposal of the brine from the oil fields has been direct discharge to streams; this causes considerable poln. of the North Canadian R., the content of chloride in one stretch of the river varying from 17,500 to 37,400 ppm. Tests made during the constr. of the new South Side sewage-treatment plant indicated that the alluvial deposits, formerly used as an emergency source of water supply, are completely saturated with brine. In addition the content of chloride in the effluent from the sewage-treatment plant renders it unsuitable for use for industrial or irrig. purposes. The U.S. Public Health Service, investigating the condition of the North Canadian River, stated that the best method of disposal of oil-field wastes is discharge into underground strata other than those producing fresh water, or storage in evapn. ponds with controlled discharge to streams.—*WPA*.

Water Pollution by Industry. A Survey of State Legislation and Regulations. D. F. OTHMER, M. D. WEISS, & R. S. ARIES. Mech. Eng., **73**:706 ('51). Poln. is defined and the public policy of the states relative to poln. is summarized. The states are showing a more realistic approach to poln.-control programs, and industry is showing a desire to cooperate with the states in solving control problems. The federal government is taking an increasing part in controlling poln. by advising local agencies, and the poln.-control program is proceeding on a more uniform and purposeful basis than during previous decades.—*CA*.

Consumption of Oxygen by Stable Organic Substances in Natural Waters. B. A. SPOPINTSEV. Gidrokhim. Materialy (Russian), **16**:61 ('49). Oxidation curve of river waters during long-term (180 days) storage in

(Continued on page 64)

(*Continued from page 62*)

small tubes at 16–20 C has a logarithmic shape; it can be expressed by a reaction equation of the 1st order. With time, oxidation becomes very small and almost const., as shown by nearly straight section of the curve. Calcd. K_D was 0.01–0.06. Initial consumption corresponds chiefly to oxidation of unstable org. compds.; very small and practically const. consumption observed after 40–50 days corresponds to oxidation of stable org. compds. Available data for short-term storage of sea water indicate curves analogous to those for river waters. Calcd. K_D was less than 0.10. Results obtained in 5–10-day tests can be used only for relative evaluations, particularly of content of unstable org. substances. For calcn. of O_2 used in oxidation of stable org. compds., storage of over 1–1.5 months is necessary. Further expts. are necessary to confirm this. Twenty-three references.—*CA*.

GROUND WATER

Ground Water Supply for New Britain, Connecticut. GEORGE W. WOODS. J.N.E.W.W.A., **65**:143 ('51). The original supply was surface water. Efforts to get a satisfactory ground-water supply are described. No trouble has been experienced with the water's quality. A table shows water characteristics from two points.—*CA*.

Economic Aspects of Ground Water in Florida. V. T. STRINGFIELD & H. H. COOPER JR. Trans. Am. Inst. Mining Met. Engrs., Tech. Pub. No. **3083-H** (in Mining Eng., **3**:525 ('51)). Ground water is one of the most valuable of Florida's minerals. There are about 327 public water supplies in Florida that serve 100 or more people, and of these about 325 obtain their water entirely from wells. 46 references.—*CA*.

(*Continued on page 66*)

(Continued from page 64)

Geology and Ground Water Resources of the Honolulu-Pearl Harbor Area, Oahu, Hawaii. CHESTER K. WENTWORTH. Board of Water Supply, City and County of Honolulu, Honolulu, Hawaii, 1951. Surveys of the geography, geology, drillholes and excavations, ground water hydrology, and the recommended constr. and research projects for this area are given.

Ground Water Resources of the Arkansas River Flood Plain Near Fort Gibson, Muskogee County, Oklahoma. STUART L. SCHOFF & EDWIN W. REED. Okla. Geol. Survey Circ., No. **28**:1 ('51). Analyses of 28 samples are given. Most are moderately hard $Ca(HCO_3)_2$ waters.—*CA*.

Ground Water Resources of the Valley-Fill Deposits in the Pittsburgh Area. D. W. VAN TUYL. Proc. Penna. Acad. Sci., **24**:155 ('50). Discussion includes the avg. compn. of the well water, and a tabulation of the industrial use of the ground water in the region.—*CA*.

Ground Water Resources of the Paintrock Irrigation Project, Wyoming. FRANK A. SWENSON, W. KENNETH BACH, & HERBERT A. SWENSON. U.S. Geol. Survey Circ., No. **96**:1 ('51). Analyses of 10 waters are given. They range from soft to very hard waters. Two contained 2.8 and 3.2 ppm F.—*CA*.

Conservation and Protection of Underground Water. Technical and Legal Aspects. International Water Supply Assoc. Congress, Amsterdam, 1949, Report **V**: 573, and Proceedings,

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165. In introducing the report on the conservation and protection of ground water, R. Brunotte discusses technical and legal measures for the prevention of poln. and describes experiences in artificial replenishment of aquifers. The report is based on information received from Canada, France, Great Britain, the Netherlands, Sweden, Australia, Italy, and New Zealand. Extracts of the reports submitted by the various countries are given in appendices.—*WPA.*

Hydrochemical Facies of Ground Water Types and Their Distribution. G. A. MAKSIMOVICH. Doklady Akad. Nauk S.S.E.R. (Russian), **56**:625 ('47). Hydrochem. facies is defined as that part of a ground water basin or stream having the same hydrochem. characteristics. The following classification is given for the zonal distribution of the ground waters over the earth: zone 1 of the tropics and subtropics with predominantly SiO_2/HCO_3 facies, zones 2 and 3 of the desert belt with predominantly Cl facies, zones 4 and 5 of the steppes with predominantly SO_4, Na, and HCO_3-Na facies, temperate zones 6 and 7 with predominantly HCO_3-Ca facies, and tundra zones 8 and 9 with predominantly SiO_2 and HCO_3-SiO_2 facies. Studies in the region of Molotovsk showed that as the ground water flowed downward the HCO_3-Ca facies was gradually changed into the SO_4-Ca-Cl facies and still farther down into the Cl-Ca-SO_4 facies.—*CA.*

Hygienic Evaluation of Means of Supplementing and Protecting Larger Ground Water Supplies. MAXIMILIAN KNORR. Gas-u. Wasserfach (Ger.), **92**: 10: 104, 12: 151 ('51). Infiltration schemes by using basins with a filter-type bottom are preferred over bank infiltration as a means of supplementing ground-water supplies. Under suitable conditions, water having a taste or odor becomes entirely satisfactory after a period of 80 days or so after infiltration. Numerous case histories are given relative to protective areas around water works and the flow of polg. materials. This flow is often very difficult to detect by chem., bacteriol., or dye test (fluorescein or uranin) as the flow may be through a limited area and normally at low rate, but greatly increased in time of flood or heavy withdrawal of water. Each case must be studied individually. Frequently it is not possible to keep the protective zone in woods, etc.; in such cases sports areas, parks, etc., may be used, with proper sanitary precautions. —*CA.*

Tracing the Underground Flow of Water. RICHARD L. DOAN & EDWIN FAST. U.S. Patent 2,553,900 (May 22, '51). Strong solns. of borax, or other water-sol. boron compds., are added to underground flows. The boron is detected by emission spectrography, by using the lines at 2495 to 2499 A. The process is used to follow water flooding of oil fields, or to trace possible contaminants.—*CA.*

HEALTH AND HYGIENE

Survey of Literature Relating to Infant Methemoglobinemia Due to Nitrate-contaminated Water. G. WALTON. Am. J. Pub. Health, **41**:8: 986 ('51).—This is an excellent survey of the literature relating to infant methemoglobinemia resulting from nitrate in water. The history of the subject is first considered. The formation of methemoglobin in animals fed with nitrates, the conversion of nitrates to nitrite in the bowel under certain conditions, and the occurrence of methemoglobinemia in patients deliberately dosed with nitrates were

(*Continued on page 70*)

(Continued from page 68)

observations made before clinical evidence was obtained that the condition in infants followed the ingestion of water with a high content of nitrate. Literature on the medical aspects is surveyed, including the causes, susceptibility, physiological effects, diagnosis, treatment, and prevention. Preventive measures consist in the provision of better water supplies after chemical survey of the existing sources in respect of their nitrate content, and in the education of the medical profession and the laity on the danger of using nitrate-containing water for making up artificial feeds for infants in their first few months of life. Recent opinion is that the nitrate content of water should be not higher than 10 or possibly 20 ppm. The APHA Committee, '49 to '50, noted over 278 cases with 39 deaths in America but point out that there are probably many more cases since the condition has not yet been made notifiable.—*BH.*

Well Water and Blue Babies. C. R. Cox. Ann. Rept. N.Y. State Assoc. Milk Sanitarians, **24**:95 ('50). There may be a relationship between the nitrate content of the water and predisposition of babies to methemoglobinemia. Public-health workers and pediatricians should accordingly view with suspicion water having 20 ppm of nitrate. Some well waters are encountered having nitrate contents of 50 to 97.4 ppm.—*CA.*

Nitrate in the Ground Water of Texas. William O. George & Warren W. Hastings. Trans. Am. Geophys. Union, **32**:450 ('51). About 3000 of the 20,000 nitrate detns. made of water from wells in Texas showed more than 20 ppm of nitrate. The public water supplies of 27 Texas towns and cities contained more than 50 ppm of nitrate. Recent medical research indicates that methemoglobinemia or infant cyanosis may be caused

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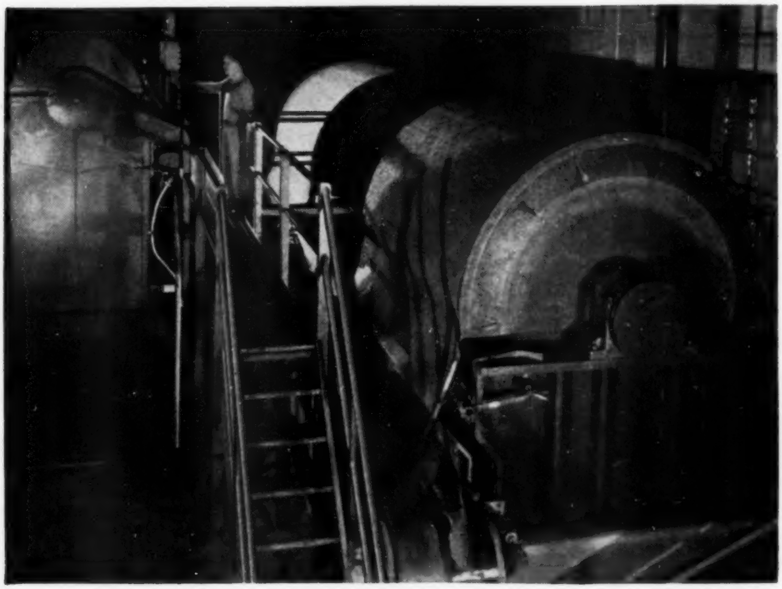

Two Eimco Filters in Sewage Plant, Waterbury, Conn. Plant construction directed by H. C. Whitlock, supervised by W. N. Kunsch.

(*Continued from page 70*)

by nitrate in the water used in formula mixts. Most of the high nitrate in ground water is found in wells less than 200 ft deep and mainly in water from late Tertiary and Quarternary formations; however, high nitrate was found in water from all kinds of rocks of all ages and appears to be unrelated to rainfall, geography, or cultivation. —*CA*.

Transmission of Disease by Water. C. R. TRASK. Munic. Util. (Water & Sanit.), **88**:11:25, 39 ('50). The role which pold. water supplies play in transmitting disease, especially typhoid, cholera, and dysentery, is outlined. Examples are given of outbreaks of these diseases traced to infected water supplies and the causes of poln. are analysed. Other diseases which may be waterborne are discussed, including gastro-intestinal infections and poliomyelitis. It is stressed that rigid and constant control is necessary over the treatment of public and private water supplies and over the disposal of sewage.—*WPA*.

Diseases of Man Spread by Water. H. MOOSER. Monatsbull. Schweiz. Ver. Gas-u. Wasserfach. (Ger.), **31**: 142 ('51). The author surveys the various types of diseases which may be spread by water, and describes some of the classical epidemics from which the relations between water and disease were discovered.—*WPA*.

The Necessity for Adequate Separation of a Process Water Supply from the Main Water Supply in Factories Using Separate Supplies Illustrated by a Report on Cases of Poisoning Caused by Waste Waters Containing Carbide Entering the Main Distribution System. J WÜSTENBERG. Arch. Hyg. Berl. (Ger.), **132**:227 ('50). Owing to cross-connections between drinking water and untreated water mains, river water containing carbide sludge entered the distr. system for the main supply and caused poisoning, in some cases fatal, in infants and young children.—*WPA*.

DISINFECTION

Use of the Bactericidal Action of Ultraviolet Radiation for the Sterilization of Drinking Water. V. F. SOKOLOV. Izvest. Akad. Nauk S.S.S.R., Otdel Tekh. Nauk (Russian), **1951**:360. Sample calcns. are presented for designing a pilot purification plant. Low-pressure A-Hg lamps are the most efficient available, but still require 10–30 watt-hr to reduce the bacterial count of 1 cu m of water to 1000/l. A 150 cu m/hr plant was operated successfully with both immersed and external lamps.—*CA*.

(*Continued on page 74*)

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Disinfection of Drinking Water on Ships by Hypochlorite or Chloramine. H. B. G. BREIJER. Ingenieur (Neth.), **63**:G55 ('51). Drinking water was chlorinated (1.25–1.4 ppm) with either "Staboclor" (I.C.I.) or "Pittchlor" (Pittsburgh Plate Glass Co.) and filtered through active C; or, NH_2Cl was produced in the water (2.5 ppm) by addn. of NH_4Cl (2.5 ppm) and "Pittchlor" (5 ppm). The second method gives a more nearly sterile water, with better taste, and is technically more simple.—*CA*.

Comparison of the Bactericidal Action of Chlorine and Ozone and Their Use for the Disinfection of Water. J. P. BUFFLE. Tech. sanit. munic. (Fr.), **45**:74 ('50). Preliminary expts. were made on the disinfection of water by ozone and by sodium hypochlorite. Samples of suspensions of pure bacterial cultures, unfiltered water from the river Rhone, and water from the lake of Geneva after passage through sand filters were exposed for various periods to different concns. of ozone and of chlorine and were then examined bacteriologically. Results of the expts. are given in tables. The same bactericidal efficiency was obtained with equal concns. of ozone and chlorine but the action of chlorine was considerably slower and was affected by factors such as temperature and pH value. After reviewing the advantages and disadvantages of the two methods of disinfection the author recommends the use of ozone which is more costly than chlorination but has no harmful effects on humans. Some papers on the bactericidal action of chlorine and ozone are discussed briefly.—*WPA*.

A New Process for the Disinfection of Water with Ozone. H. SCHMASSMANN. Tech. Rdsch. (Swiss), **43** ('50). The water supply of Füllins-

(Continued on page 76)

(*Continued from page 74*)

dorf, Switzerland, is obtained from ground water in the Ergolz valley, and in recent years the pollution of this ground water has increased to such an extent that it has become unsafe to drink the water unboiled. In September '50 a plant for disinfecting the water by ozone was placed in operation. In this plant ozonized air is prepared by drawing air between an inner electrode consisting of an enamelled steel tube and an outer electrode made from a special type of glass; a voltage of 10,000 v is developed between the electrodes. The ozonized air is cooled and then compressed and released as bubbles in the base of a column of the water 5 m deep. As well as improving the hygienic quality of the water, treatment with ozone has improved its chemical quality by removing aggressive carbon dioxide.—*WPA*.

Chlorine Dioxide as a Bactericide for Water Treatment. R. S. Ingols. J. Inst. Wtr. Engrs. (Br.), **4**:581 ('50). The author summarizes recent literature on the disinfection of water with chlorine dioxide. It has been shown in laboratory studies that *Esch. coli*, many intestinal pathogenic organisms, the virus of poliomyelitis, and spores of several strains of bacteria are sensitive to low concns. of residual chlorine dioxide. The spores of many bacteria are much more sensitive to chlorine dioxide than to chlorine, even when water is chlorinated beyond the break point. In the concns. required for the disinfection of water, chlorine dioxide generally does not produce objectionable tastes and odors. At pH 8.0 it is very stable. The chemical characteristics of chlorine dioxide and methods for its detn. are described.—*WPA*.

(*Continued on page 78*)

(Continued from page 76)

Sterilization With Chlorine Dioxide of Water at Niagara Falls, N. Y. F. V. H. Piper & J. Burnett. Wtr. & Sanit., **88**:6:24, 44 ('50). The use of chlorine dioxide in preventing the development of tastes and odors in water in the distribution systems of Niagara Falls, N.Y., is discussed. It appeared that secondary tastes and odors were formed by reaction of free chlorine in the treated water with organic matter present in the mains. Previously, chlorophenolic tastes and odors in the raw water had been satisfactorily removed by chlorine dioxide and tests were made to determine whether presence of chlorine dioxide and absence of free chlorine in the treated water would give water of satisfactory bacteriological quality and also prevent the development of secondary tastes and odors. Treatment at the water works comprises coagulation, sedimentation, and rapid sand filtration, and facilities are provided for preliminary and postchlorination. Results have indicated that by preliminary treatment with chlorine to give only a trace in the filter effluent and by maintaining a residual content of chlorine dioxide in the water entering the distribution system, water of satisfactory quality is obtained and there is no apparent deterioration in the distribution system.—*WPA.*

OTHER ARTICLES NOTED

Recent articles of interest, appearing in American periodicals, are listed below.

Spectrochemical Analysis of Radioactive Solutions. C. Feldman, M. B. Hawkins, M. Murray, & D. R. Ward. Anal. Chem., **22**: 1400 ('50).

Potomac Valley Conservancy District Takes the Offensive Against Pollution. H. A. Kemp. Civ. Eng., **20**:17;70 ('50). Acct. given of sources and extent of poln. of the Potomac R. and work of the Interstate Commission on the Potomac River Basin in controlling poln. described.—*W.P.A.*

The Reading Meter

(Continued from page 22)

Cooling Towers. *J. Jackson. Butterworths Scientific Publications, London (1951) 104 pp.; $3.50 from Butterworth & Co. (Canada) Ltd., 1367 Danforth Ave., Toronto 6, Ont.*

This monograph, dealing principally with mechanical-draft water-cooling towers, has been published under the auspices of Imperial Chemical Industries Ltd. to remedy what the author terms a lack of adequate published information on the subject. In addition to design factors, attention is paid to the economic alternatives of recirculating water through cooling towers, obtaining water from the municipal supply, or pumping from a river or other body of water.

MAPI Accounting Manual. *Machinery & Allied Products Inst., 120 S. La Salle St., Chicago 3, Ill. (1952) $15.*

Designed for the machinery, industrial equipment, and capital goods producing industries, this volume nevertheless offers analyses of accounting problems common to many industries today. Some of the most salient concern methods of computing depreciation and inventory values—such as the substitution of replacement value for original dollar cost and the adoption of the "last in, first out" procedure for inventory evaluation. Although not recommending specific measures, the book does suggest the review of financial accounts in terms of economic income and values, and strongly hints that such a review may show the need for improved accounting methods.

A wrapping machine for applying plastic adhesive tapes to small diameter pipes is available from Plastic Engineering & Sales Corp., Box 1037, Forth Worth, Tex., together with supplies of corrosion-resistant tape. Known as the Pesco Tapester 5, the manually powered machine is said to have a wrapping speed of 15 to 30 fpm and to be able to ride over field joints, welds and couplings.

A sound film showing the cleaning of Kansas City's sewers during the disasterous 1951 flood has been prepared for showing to interested groups by Ace Pipe Cleaning Contractors, Inc., 2003 Indiana Ave., Kansas City, Mo. The film was professionally prepared and edited and brings reality to the complicated action, which was taken under the stress of disaster.

Upsadowsing

To the Editor:

Ater all of the comment in Percolation and Runoff, it appears that the attached cartoon series, clipped from *Diario de Occidente*, one of our local newspapers, might offer further possibilities for research.

HOWARD D. SCHMIDT

Div. San. Engr.
Creole Petroleum Corp.
Maracaibo, Venezuela
Mar. 29, 1952

EL TIO DE LAS BARBAS

Taking friend Schmidt at his word rather than his thought, we channeled some immediate research into "Uncle Whiskers" antecessors and came up with the dowdy dowser pictured below from "This Week" magazine of Nov. 19, 1950. All of which proves something about something else than dowsing. With things looking up this way, we'll let the rainmakers take it from there.—ED.

(Continued from page 20)

Slight limp in the march of science must have been the discovery that white rats given a series of electric shocks will drink 27 per cent more water than they do normally. Even if, in the end, the finding is ponderously pronounced applicable to humans, how many water works men these days have supplies adequate enough to go in for that type of sales promotion? Especially useless does the information seem in the light of a simultaneous discovery that approximately one-third of the rain produced in an average thunderstorm never reaches the ground, evaporating before it gets there. As if we haven't supply problems enough without scientifically seeking more.

Coming attraction that has our mouth—yes—watering is a new novel by Henry Morton Robinson, to be called "Water of Life." What if the subject is whiskey in all its moral, social, financial, and theological aspects, we can kid ourselves into thinking it's the title we like.

William G. Riddle has joined the Kansas City, Mo., consulting firm of Charles A. Haskins. He had previously been with Burns & McDonnell Engineering Co. and with the Oklahoma A. & M. College.

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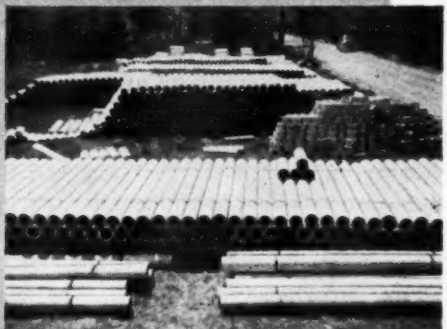
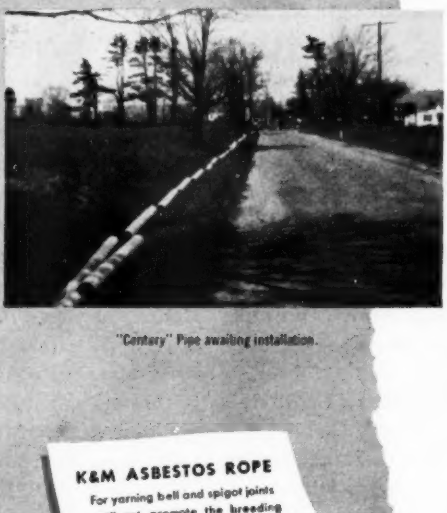


Storage of "Century" Pipe and "Century" Simplex Couplings is shown here on site in New England preparatory to installation.

"CENTURY"

ASBESTOS-CEMENT PIPE

gives a New England town 19 miles of trouble-free water mains



"Century" Pipe awaiting installation.

Thus, another community is added to the growing and already large number of places in every State where this modern water main has been adopted.

Right from the start this modern water system provides economical operation by the relatively low cost of installation and reduced carrying charges plus the advantages inherent in the pipe itself.

Made of two practically indestructible materials, asbestos fiber and portland cement, "Century" Pipe is strong and durable. It is highly resistant to external corrosion and internal corrosion (tuberculation) cannot take place due to its non-metallic composition. The permanently smooth inner surface guarantees continuous flow and minimum pumping costs.

It will pay you to investigate the many cost saving features of "Century" Asbestos-Cement Pipe.

K&M ASBESTOS ROPE

For yarning bell and spigot joints will not promote the breeding of bacteria. Write for details.

WRITE FOR FREE BOOKLET, "Mains without Maintenance". Gives valuable data, specifications, and reference material for anyone interested in water main pipes.

Nature made Asbestos . . .
Keasbey & Mattison has made it serve mankind since 1873

K&M best in asbestos

KEASBEY & MATTISON
COMPANY • AMBLER • PENNSYLVANIA

(*Continued from page 84*)

The bitter cups of Mt. Clemens, Mich., that distastified the water served in them at a local soda fountain (see February P&R p. 78) turned out to be bitter because of a phenolic disinfectant used by the paper manufacturer in controlling slime on the screens employed in draining the pulp. The cup manufacturer discovered this only after a thorough investigation of the printing ink, the wax, and the glue used in his processes.

Meanwhile, Mt. Clemens water caused more and louder sputtering at the local USO, where servicemen—not noted for their fondness for the stuff under the best of circumstances—wouldn't touch the water that gushed out of the new electric cooler. Super Bob Hansen, by then a supersnooper as well, went to work on the case forthwith, and in no time at all traced the machine from factory, to service department, to plumber, who admitted using a castor-oil base pipe dope in making the installation. A minor cleanup job and the brew was palatable again.

It just goes to show you, though, all the bugs in water supply sure aren't bacteria.

H. O. Proske has been appointed service manager for Rockwell Mfg. Co., and will establish product repair and parts handling procedures in factories and warehouses of the company. He has been with Rockwell since 1937 as a sales engineer in the Kansas City region.

Imperialism on a local scale was the accusation when a meter reader of the Caldwell, N.J., water department recently extended his territory four houses into neighboring North Caldwell. Better than just taking over the territory, however, the eagereyed one managed it so that North Caldwell provided the supply that Caldwell collected for. What if the dastardly deed was discovered, what if Caldwell did give up its loot, and what even if the fellow had been fired? We know lots of water departments, companies, even countries who could use a man as ambitious and effective as all that.

Even at the risk of ruining the story, we ought to explain that a year before the discovery, North Caldwell had built its own water system and discontinued purchasing a supply from Caldwell. It was immediately following the changeover that the reader misundertood his new instructions and read the meters in four houses that were actually served by the new North Caldwell system. Not wanting their meters overread, householders shooed away the North Caldwell man who came shortly thereafter, and that continued that.

With restitution made, we wonder if North Caldwell is paying Caldwell for meter reading rendered.

(*Continued on page 88*)

(*Continued from page 86*)

Federal aid amounting to $356,000 has been granted to the Cobb County-Marietta Water Authority in Georgia to construct water works. The facilities are to serve areas that have been classified as a critical defense housing area, and the grants were made by the Public Health Service and the Housing and Home Finance Agency.

William D. Wheatley, chief chemist for the Pittsburgh Plate Glass plant at Crystal City, Mo., died on March 13 of a heart ailment after a brief illness. As chief chemist for the company's Works No. 9, Wheatley not only had charge of the annealing of glass and other industrial operations, but also had charge of the plant's and Crystal City's water supply. A member of AWWA since 1945, he had served as vice-chairman and chairman of the East-Central Section of the Missouri Water and Sewerage Conference.

"Worthington Corp." is the new name for the century-old Worthington Pump and Machinery Corp. Organized in 1940 to build a new type of steam pump, the change in name reflects the fact that the organization's interests are now greatly diversified, with pumps representing only 30 per cent of the volume of business.

(*Continued on page 90*)

(Continued from page 88)

Moose juice was our reaction when we first heard of Goshute-Antelope Valley in Elko County, Nevada. But from the evidence of our own abstraction (April Condensation, p. 78 P&R) we have to confess not only that there is such a valley in such a county in Nevada, but that it contains ground water and that the ground water contains "sulfate, Cl and F." Actually at a season like this, even if it were to rain deer in the valley, we'd be more interested in Goshute-Par!

Speaking of the season, though, the Southeastern Section identifies it in different terms, as witness the quarantine notice which was prominently posted at its recent meeting in Augusta, Ga.:

WARNING!

FISHING FEVER

VERY CONTAGIOUS TO ADULT MALES, ALSO FEMALES

SYMPTOMS–Continual Complaint as to Need for Fresh Air, Sunshine and Relaxation. Patient Has Blank Expression, Sometimes Deaf to Wife and Kids. Has No Taste for Work of Any Kind. Frequent Checking of Tackle Catalogs. Hangs Out in Sporting Goods Stores Longer Than Usual. Secret Night Phone Calls to Fishing Pals. Mumbles to Himself. Lies to Everyone. NO KNOWN CURE.

TREATMENT–Medication Is Useless. Disease Is Not Fatal. Victim Should Go Fishing as Often as Possible.

WILEY BASS, Dept. of Public Health.

Meanwhile, the Brooklyn Chapter of the New York Section persists in its affiliation with the Flatbush Society for More Frequent Grandmother Interment—Ebbets Field Contingent.

It happens every spring!

Every spring, too, the misdistribution of water in both time and space begins to make itself more felt. With the Midwest all aflood, Texas was wishing, working, even praying for rain. The level of Lake Erie was up some 18 inches, causing considerable damage to lake-front homes, highways, and seawalls. Similarly, the Salton Sea pulled itself up a couple feet

(Continued on page 92)

(*Continued from page 90*)

to 237 below sea level and forced relocation of runways and other facilities of the Sandy Beach Atomic Energy Base. And New York City, with reservoirs brimming, began worrying again about squandering now what it will need later. Too much, too little, too soon, too late, or even just right —as in the Salt River Valley last April—for too short a time.

The Texas thirst has all the earmarks now of another well-told tale of drydom. "Today is a fine day," noted a radio commentator, "it is raining in many parts of Texas." Meanwhile, the daily headlines tell of a prayer meeting that successfully brought a shower to Del Rio; a drizzle at Abilene; showers in "upper south Texas"; rainmaking experiments in 26 counties; delivery of water from Mexico's Marte R. Gomez Reservoir under agreement with the International Boundary and Water Commission; reuse of air-conditioning supplies at San Antonio; water rights at San Angelo. One of these days some New Yorker with elephantine enough memory to recall the drought of '49 is going to repay that tanktruckful that Texans contributed when the roles were reversed (Feb. 1950 P&R p. 2). And some one else from somewhere else is sure to point out that even in Texas water and oil can't be expected to mix. Even without stunts, though, waterlessness, if not water, is getting more than its share of the headlines, as the drought, if not the thirst, goes on.

Victor A. Hann has been elected executive vice-president and member of the board of the Welsbach Corp., effective April 1. As director of the Ozone Processes Div. of Welsbach Corp., Hann has been working on plans for an ozone installation for Emery Industries of Cincinnati that will be the world's largest.

The 1952 Gordon Research Conference on Ion Exchange will be held July 7–11 at the New Hampton School, New Hampton, N.H. The conference is one of the group of Gordon Research Conferences sponsored by the American Assn. for the Advancement of Science and will include papers on ion exchange counter current systems, kinetics, electrodes, membranes, and economics. Details may be obtained from W. George Parks, director of the Dept. of Chemistry, University of Rhode Island, Kingston, R.I.

M. A. Matthews, sales engineer for Neptune Meter Co. in the Houston, Tex., area, has been recalled to active duty with the Marine Corps. He is currently stationed at Camp Lejeune, N.C., as a major.

(*Continued on page 94*)

(*Continued from page 92*)

The tank collapse at Tucumcari, N.M. (see this issue, p. 435), has now resulted in damage suit claims totaling $120,000. Four persons were killed and six injured when the municipal water tank gave way last December 13. The suit claims that the city was negligent in erecting and using the tank.

A laboratory-size deionizing kit has been devised consisting of a polyethylene bottle that can be filled at the tap and squeezed to force the water through a special blend of ion exchange materials. The Deeminac, as it is called, is said to yield up to 20 gal. of water equivalent to the triple-distilled product on a single charge of chemical. Recharge filters are available and must be used when the change in color of the original indicates that it has been exhausted. A circular is available from E. Machlett & Son, 220 East 23rd St., New York 10, N. Y.

Morris M. Cohn, sanitary engineer of Schenectady, N.Y., and editor of the Case-Shepperd-Mann publication, *Wastes Engineering,* has been appointed city manager of Schenectady. He will retain his editorial post.

(*Continued on page 96*)

(Continued from page 94)

Interutility cooperation and interdependence will be featured by an unusual combination of water treatment plant and steam power generator to be constructed at Fairbanks, Alaska. Water from the supply wells will first be used by the power plant for condenser cooling. The effluent water, having been warmed to 60 F, will then be processed by the water plant and pumped into the distribution system, with, engineers feel, much less likelihood of freezing. The 1.4-mgd plant is said to be the first in Alaska to provide complete water treatment.

Thar's gold in them thar rills—the ones, at least, that feed the Chattahoochee River; and all that gold, picked up by the water in flowing through "some of the richest gold country in the world," is what one Benjamin P. Tuggle, mining engineer of Atlanta, Ga., is after. Having found that the river assayed between five and seven cents per ton, Mr. Tuggle spent the last fourteen years perfecting a machine to recover them pennies before they went to sea. Just patented, the Tuggle "Splash It Rich" device is based on the principle of the centrifuge, spinning gold particles into a mercury lining inside a separator unit through which the watore is passed. Admittedly impractical in small-scale installations, the machine is scheduled for a big-time operation—a bank of 100 machines, costing half a million dollars, handling 3,000 tons of water and silt per minute, and netting approximately $45,000,000 per year. Only trouble right now is half a million dollars, difficulty in obtaining materials for building the device, and a more immediately profitable mica-mining business. But what price water after a Korean truce? Our reservoirs will be all banks, and even if we don't mine them, we ought to be able to get more for a supply that's recognizably enriched.

A. L. Jones, formerly manager of the water treating and deaerator section of Worthington Pump & Machinery Corp., has been appointed manager of the Water Treating Div. As such he will be in charge of sales, engineering, production and service of water treatment equipment. He has been associated with Worthington since 1920.

Leonard A. Scheele was sworn in for his second term as Surgeon General of the Public Health Service on April 3. The Senate had earlier confirmed the reappointment. The seventh Surgeon General to be appointed since Congress created the post in 1870, Dr. Scheele has been with the service since 1934, and was only 40 years old at the time of his first appointment as its head in 1948. During the war he served in the Mediterranean Theater and Italy, and in 1944 was medical member of General Eisenhower's planning staff.

(Continued on page 98)

(*Continued from page 96*)

A "plumber's helper" ain't what it used to be, at least not in Oradell, N.J., since master plumber Walter Fischer had an idea. Now the term means "everybody's wife," young, old, or unadmitted, for almost everybody's wife is attending Fischer's free classes in plumbing repairs for women.

Noting the increase in unprofitable "change a washer" calls from new housing developments in the suburban community, Fischer decided it would be good business, not only for him, but for all plumbers to teach homeowners what to do themselves and what to call a professional for. He picked on the women, first because they are closest to the troubles and, second because the men didn't respond when they had a chance. What he may not realize, of course, is that the men actually are responding in their own way and to their own best advantage. Be that as it may, water works men must be right with Fischer's fellow plumbers in enthusiasm—professionally, too, for the water it ought to save from husbandial procrastination.

Typical tips of a Fischer lecture-demonstration are: "Use a hard washer, not a soft rubber one, on a hot water faucet." "Use the right tool for the job." "Always tackle the largest nut when taking a faucet apart—usually it is the one that should come off first."

Wonder who stops the drips in the Fischer household?

(*Continued on page 100*)

(Continued from page 98)

Fluoridation fact and fable continue to accumulate—fact, as usual, like a tortoise and fable like a hare. From Madison, Wis., however, came the eagerly awaited first report on experience with a fluoridation chemical other than sodium fluoride. A study in the west area of the city by the Madison Dept. of Public Health and the Dane County Dental Society after 3½ years addition of hydrofluoric acid indicated even better results than those reported by Brantford, Ont., and Sheboygan, Wis., after similar periods of experience—a reduction of 67 per cent in average decayed surfaces per five-year-old kindergarten child compared with those in the control city, Stevens Point, Wis. And from *Lawn Care,* publication of O M Scott & Sons Co. of Marysville, Ohio, came reassurance, based on both experience and experiment, that fluorides in lawn sprinkling water would not hurt the grass, intimations to the contrary from Seattle notwithstanding. Negative fact being even scarcer than the positive these days, we've been unsuccessful in this month's sift, but we keep straining. Meanwhile, the stuff we sift seems so much more productive of psychological than physical fact that we're tempted to change the subject.

Henry T. Sulcer, general manager of Graver Water Conditioning Co., has been elected vice-president of the parent organization, Graver Tank & Mfg. Co., of East Chicago, Ind. Prior to coming to New York to head the subsidiary firm, Sulcer had been auditor for all divisions of Graver.

James L. Baumgartner has joined the sales force of Neptune Meter Co., and will represent it in the Houston, Tex., area.

The sea water desalting bill, on which hearings were completed last fall, has finally been reported out of the House Interior Committee. Known as HR 6578, the bill would authorize research into the conversion of sea water into fresh water, with some practical procedure as its goal. The demonstration plant called for by the Engle bill has been eliminated, however, and the $1,000,000 appropriation required by the earlier version has been spread over five years.

Peter Ley, Arthur McCauley, and Maurice V. Hegarty have been elected vice-presidents of the Jamaica, N.Y., Water Supply Co. Ley, who started in the utility's Engineering Dept. in 1924, has also been made chief engineer. McCauley, a member of the bar in New York, has been secretary and attorney for the company for 17 years. Hegarty, who has been with the firm 20 years, also becomes chief of operations. In other actions taken by the company's Board of Directors meeting in April, Mrs. Katherine Leslie was reelected chairman and D. J. Hennessy was reelected president.

Resistance-to-ground measurements may be made by Vibroground instruments, according to a circular distributed by Associated Research, Inc., 3758 W. Belmont Ave., Chicago 18, Ill. In addition to the model designed for quick determination of the resistance to earth of man-made grounds, the company offers instruments for geophysical prospecting by the four-point method, for direct-reading soil resistivity measurement for corrosion surveys, and other purposes.

Main cleaning services offered by Ace Pipe Cleaning Contractors, Inc., are described in an illustrated folder available from the company. The company uses hydraulic pressure scraping tools capable of cleaning mains from 4 to 60 in. in size. For extreme incrustation, power driven tools are used. A sewer cleaning service is also described. The firm is located at 2003 Indiana Ave., Kansas City, Mo.

Mechanical shaft seals are featured in a new line of horizontal, split-case, general service pumps in discharge sizes from 1½ to 4 in. The capacity range is up to 750 gpm, with a head range up to 230 ft. Replacement of the conventional stuffing box by the standard mechanical shaft seal is claimed to save floor space, eliminate shaft leaks, permit easier servicing and shorter shaft lengths, and improve performance characteristics. The details are given in a bulletin No. B-1350, on the Type AS pump distributed by Peerless Pump Div., Food Machinery & Chemical Corp., 301 West Ave. 26, Los Angeles 31, Calif.

Centrifugal pump instruction booklets have been released by Allis-Chalmers Mfg. Co., 1026 S. 70th St., Milwaukee, Wis., and are available on request. Bulletin 08X7780 contains 48 pages of installation, operation, and repair instructions for multi-stage centrifugal pumps. Bulletin 08X7813 is just half as long and covers single stage-single suction pumps. Both bulletins include information on unloading, aligning, special features like the "Magic-Grip" coupling, and a table of trouble-finding tips.

Loss-in-weight gravimetric feeders for dry materials are described in a new folder, 30-H12, issued by Omega Machine Co., 345 Harris Ave., Providence 1, R.I. In addition to describing the operation and components of the equipment, the bulletin describes such accessories as chart recorders, hopper agitators, and dissolving chambers.

Speed reducers of the double-worm and helical-worm gear types are described and their specifications detailed in a 48-page bulletin available from De Laval Steam Turbine Co., Trenton 2, N.J. Information is also provided on how to select double reduction gearing, and horsepower rating tables and dimension sheets are included.

Air-operated butterfly valves offered by Minneapolis-Honeywell Regulator Co., Wayne & Windrim Ave., Philadelphia 44, Pa., are described in a 12-page bulletin available from the company. The assemblies feature Continental butterfly bodies and Honeywell Air-O-Motor diaphragm operators.

Water conditioning units, including softeners and deionizers for process and boiler waters, are described in a 20-page booklet, No. 611, distributed by Elgin Softener Corp., Elgin, Ill.

(*Continued on page 104*)

(*Continued from page 102*)

Coal-tar base enamels for protective coating of pipelines are described in a 24-page booklet available from the company. Information on a compound for protection of mechanical pipe couplings is included, as well as a table of quantities required.

Capitalism and the stock market are suggested as subjects for company house-organ features by the brokerage firm of Merrill Lynch, Pierce, Fenner & Beane, 70 Pine St., New York 5, N.Y. A booklet entitled "They're doing it—Are you?" is offered, giving examples of such features run by nationally prominent companies, and the firm offers its assistance for such preparation without charge. Reprints of an advertisement, "About This Stock and Bond Business," are also offered in quantities up to 2,500 without charge.

Dressertape, a high dielectric plastic tape for the protection against corrosion of underground pipe, is described in a circular distributed by Dresser Mfg. Div., Bradford, Pa.

Mechanically operated filter gages are described in a new folder, Bul. 450-H10, available from Builders-Providence, Inc., 345 Harris Ave., Providence 1, R.I. The operation of diaphragm pendulum units, used as head loss and flow indicators, is detailed, and specifications are given for the gages.

Flow meters for measurement of open channel flows through a Parshall flume or a weir are described in a circular, Bul. F1606, being distributed by Bristol Co., Waterbury 20, Conn. The meter operates by measuring the head of water upstream from the weir plate or flume throat and converting this factor into terms of flow.

Coming Meetings

May **26–28**—Canadian Section at Mount Royal Hotel, Montreal. Secretary, A. E. Berry, Director of San Eng., Parliament Bldgs., Toronto 2, Ont.

June **18–20**—Pennsylvania Section at Lawrence Hotel, Erie. Secretary, L. S. Morgan, Div. Engr., State Dept. of Health, Greensburg, Pa.

25—New Jersey Section Summer Outing. Luncheon at Martinsville Inn, Martinsville. Inspection of Elizabethtown's Millstone-Raritan Filter Plant.

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AWWA's 73rd Annual Conference

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Sleeves; see Clamps

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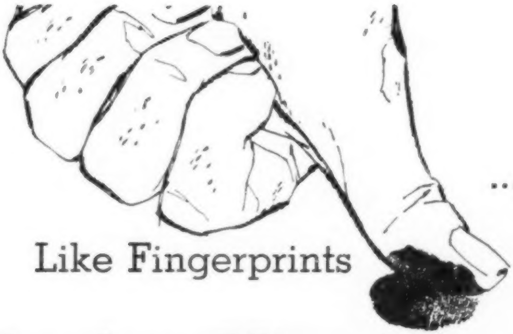
Directory of Professional Services—pp. 25-29



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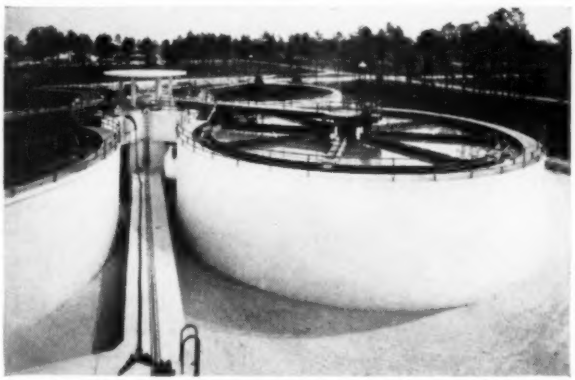

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